AN INVESTIGATION INTO THE COMPATIBILITY OF THEORETICAL DESIGN AND FIELD TESTS FOR FOUNDATIONS AT LANGLEY FIELD, VA.

A Thesis

Presented to

the Faculty of the Department of Engineering

University of Virginia

In Partial Fulfillment

of the Requirements for the Degree

Master of Applied Mechanics

by

James F. McNulty

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APPROVAL SHEET

This thesis is submitted in partial fulfillment of the requirements for the degree of Master of Applied Mechanics

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For Subcommittee

Chairman, Committee on Graduate Studies in Engineering

June 1954

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NOMENCLATURE

B	width
b	side of a square footing
c	cohesion
c_v	coefficient of consolidation
D_{10}	effective grain size
D_f	depth of foundation
e	void ratio
e_o	void ratio in natural state
f_s	skin friction on pile per unit area
H	half thickness of compressible layer
I_w	plasticity index
L	applied load
L_w	liquid limit
m_v	coefficient of compressibility
N	dimensionless bearing factor (N_c, N_γ, and N_q)
P_w	plastic limit
p	pressure or normal stress
p_v	vertical pressure against horizontal plane
Q	critical load on a pile
Q_a	allowable load on a pile
Q_d	critical load per unit length on a continuous footing
Q_{db}	critical load on a square footing
Q_{dr}	critical load on a circular footing

q	applied load per unit area
q_a	allowable soil pressure
q_d	critical unit load by a continuous footing
q_{db}	critical unit load on a square footing
q_{dr}	critical unit load on a circular footing
r	radius
s	shearing resistance
T_v	time factor
t	time
U	uniformity coefficient
$U\%$	per cent consolidation
w	water content
z	depth
γ	soil unit weight in place
γ'	submerged unit weight
γ_d	dry unit weight
δ_T	total settlement
ϕ	angle of internal friction

CHAPTER I

INTRODUCTION

To term Soil Mechanics, in its present status, a science is to twist the truth to a breaking point. Soil Mechanics is still in its adolescence and, as long as soil continues to retain its nonisotropic and nonhomogeneous characteristics, is likely never to obtain the full growth of his distant relatives, Physics and Chemistry. Soil Mechanics has, however, traveled a long road since its infancy and has marked that road with many guideposts to aid the designing engineer. It is now possible for the designing engineer, on the basis of soil tests and applicable theory, to make an approximate forecast of a foundation's load-carrying and settlement characteristics.

The primary purpose of this thesis is to present a sound design procedure for foundations to be supported by the underlying silty-sand stratum at Langley Field, Virginia. It is hoped, also, that the methods utilized to obtain the sound procedure may aid other engineers in setting up design procedures for other localities.

Secondarily, test results from both the laboratory and the field are included in order to present to the soils' theoretician experimental data to check existing and future theory. To enable the theoretician to apply the test results properly, a complete classification of the silty-sand's properties is contained in Chapter IV.

Despite the fact that many field tests are made daily in this nation, the results of only a few are made available to the soils' engineer by means of technical papers; unfortunately, the great majority of these lack the completeness necessary for correct correlation. It is realized that this thesis errs in this regard in at least one instance. This instance is the failure to load all test piles to the ultimate. It is believed, however, that this error, though important, is not critical.

The final purpose of this report is to point out and discuss the apparent discrepancies in theory and test results so that an unsuspecting engineer will not place too great a reliance upon the blind use of any formula.

The importance of each locality setting up its own design codes cannot be overemphasized. The semiempirical rules offered in design manuals and texts for the determination of safe bearing capacities and settlement estimates are based on tests on similar type soils from random locations. Since there is a great variance within a single soil type, the resulting formulas are apt to be ultra-conservative for any individual community.

It is becoming increasingly important to estimate settlement closely in the design of foundations for wind tunnels. A differential settlement, which could be tolerated in an ordinary building, could cause millions of dollars damage to a wind tunnel. It is estimated that a building can withstand a differential settlement of 3/4 of an inch between adjacent columns without damage; it is not inconceivable that such a settlement in critical portions of a wind tunnel could render all the test data worthless or cause great magnification of the forces on the drive shaft resulting in costly repair.

The increasing pace of aeronautical science yearly causes many tunnels to become obsolete. These tunnels are often returned to usefulness by repowering and other alterations. These changes usually result in a great additional weight being placed on existing footings. Since underpinning is a very costly item, it is prudent to make a detailed analysis in order to check the foundation's adequacy for the increased load. For these reasons, a knowledge of soil mechanics is especially pertinent for the designer of foundations for research facilities.

The chapters are organized in a manner such that the reader is presented all the test and theoretical data prior to the discussion and recommendations. It is believed that this arrangement gives a better over-all picture to the reader than to discuss the subject and cite test data to substantiate the recommendations. The chapters are arranged as follows: Chapters II and III furnish background by summarizing the history and citing procedure and sources of data. Chapter IV classifies the silty sand by standard classification tests. Chapters V and VI predict the load-carrying and settlement characteristics of the silty sand on the basis of laboratory tri-axial shear and consolidation tests. Chapters VII and VIII present the load-carrying and settlement characteristics of the silty sand based on actual field tests. Chapters IX, X, and XI discuss the discrepancies between laboratory predictions, field tests, and purely empirical methods. Chapter XII contains the summary, conclusion, and recommendations.

CHAPTER II

HISTORY

It is estimated that the first studies in what is now called "Soil Mechanics" originated in France about the sixteenth century. These studies were of earth pressure as applied to fortifications. An outgrowth of the studies was Coulomb's "Theory of Earth Pressure" published in 1773; it is believed to be the first paper published regarding the characteristics of soil. The next notable contribution was made by the British scientist Rankine in 1856 when he published a paper developing a theory of equilibrium of earth masses. These two theories are jointly termed the "classical theories of soil mechanics."

Prior to the twentieth century, there was not much advance in the science of soils, the reason being that the large buildings of that era were framed by massive walls which could withstand large settlements without damage. If the ground was obviously "soft," the walls were supported by piles. Despite the above, many buildings either collapsed or were badly disfigured.

Two reasons brought the need of a knowledge of soil mechanics to the foreground. The first reason was the need for economical, large buildings to house the great increase in industrial capacity; these buildings had light exterior walls which were susceptible to differential settlement. The second reason, principally in Europe, was that the scarcity of land area was forcing building in locations which had previously been avoided because of poor soil conditions.

Within a short period after the turn of the century, scientists made great strides in this fertile field. Dr. Karl Terzaghi was the foremost investigator; in 1925, he authored the first book on the subject of soil mechanics. It seemed logical at that time that a new field of exact structural analysis was just "around the corner." However, as theory and test results continued to show discrepancies, it became obvious that only approximate forecasts could be made due to the nonhomogenity of the soil.

The trend now seems to be away from precise mathematical theory and intricate testing and toward making more accurate assumptions. Better results are now obtained with simplified formulas once the mode of failure is visualized than with rigorous theory. The soils' engineer bases his visualization on study of past failures and on a study of the characteristics of various type soils.

Research work in soils is still gaining momentum in the race to narrow the gap between theory and reality. Among the leaders in various research and educational institutions specializing to a great extent in soils in this country are the following:

U. S. Corps of Engineers
Public Roads Administration
State Highway Depts. of Michigan,
California, and Virginia
American Society of Civil Engineers
Harvard University
Massachusetts Institute of Technology

The leading countries in the study of soil mechanics, in addition to the United States, are Great Britain, Russia, and Sweden.

Although there is no periodical devoted solely to soil mechanics, many periodicals accept papers on the subject. The leading source of information is the Separates printed by the Soil Mechanics and Foundation Division of the American Society of Civil Engineers.

CHAPTER III

METHOD OF PROCEDURE AND SOURCES OF DATA

There are three main types of data used in this thesis. They are as follows:

(1) Laboratory test data

(2) Field test data

(3) Theoretical formulas

Since NACA has no soils laboratory, private firms, other government agencies, and various universities have been contracted to do the laboratory test work. On most large NACA construction projects, some type of soils investigation is required. These investigations always require borings and laboratory tests; in some instances, recommendations for design procedure are requested. The author is responsible for specifying the magnitude of the investigation required, what type tests are to be made, and, finally, for checking and approving the work of the contractor. In this capacity, the following consultants have performed laboratory investigations:

Wm. S. Housel of University of Michigan

Edward S. Barber of University of Maryland

Waterways Experiment Station - U. S. Corps Engr.

Haller Testing Laboratories, Inc.

Raymond Concrete Pile Company

In his capacity of soils' specialist, the author writes the specifications for all pile foundations and other earthwork. On all large projects, test piles are specified. During the pile load testing, the author supervises the test and records pertinent data. At the conclusion of the test, the author recommends the length of pile to be driven. Test pile data obtained in this manner are included in the thesis.

Theoretical formulas and methods of analysis are obtained from texts and scientific papers. All texts noted in the references were used for this purpose but, by far, the most useful source of data was SOIL MECHANICS IN ENGINEERING PRACTICE by Terzaghi and Peck.

Briefly, the method of procedure followed in the thesis is as follows. The first step is the sifting and investigation of the laboratory tests to determine the principal characteristics of the soil. The second step involves the application of these characteristics by means of recognized theory to foundation design. The next step is to determine the soil's characteristics by investigating the results of actual field tests and to apply these characteristics to foundation design. The final step is to investigate and discuss their compatibility, or lack thereof, and to recommend a method for future foundation design.

CHAPTER IV

DETERMINATION OF SOIL CLASSIFICATION

In order for the test results and settlement data contained elsewhere in this thesis to be properly correlated with soils' theory, it is essential that the tested soil be properly identified. For this reason, complete data concerning boring records, gradation curves, liquid limit, plasticity index, and natural density are presented.

The subsoil at Langley Field, after penetrating the first 5 to 15 feet, is a fine silty sand with shell fragments extending to an undetermined depth; NACA has taken borings exceeding 100 feet without finding a change in this stratum.

In 1939, the Layne Atlantic Company, in search of artisian water to cool the generators in the NACA Generating Plant, unsuccessfully drilled two holes 500 and 700 feet deep. According to NACA's chief construction inspector, the auger was still bringing up the grey silty sand. This silty-sand stratum extends all over the lower Virginia peninsula according to the representative of the Raymond Concrete Pile Company in charge of the boring crews.

NACA has had approximately 50 well scattered borings made on the base; the boring log shown on the next page, Figure 4-1, is presented as representative. The standard sampling procedure of a weight of 140 pounds falling

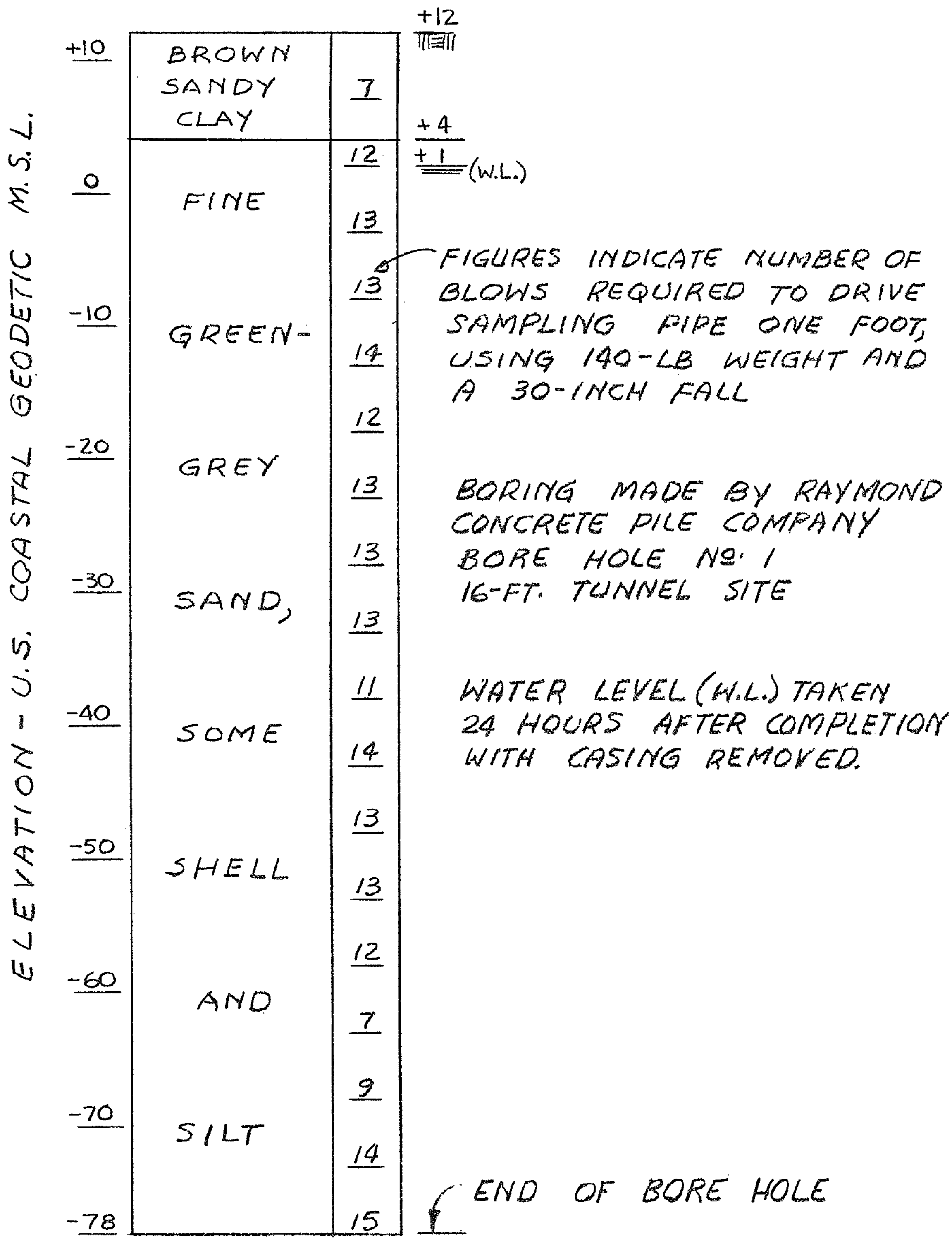


FIGURE 4-1

BORING LOG

30 inches on a 2-inch outside diameter spoon was followed; the number of blows required to drive the spoon 1 foot is designated the "Gow" penetration value. Except for minor variations in the "Gow" penetration value, all borings reveal practically the same subsoil condition once the top layers are penetrated. These upper strata show wide variance at different locations; however, they are usually less dense and highly compressible. Since the "Gow" penetration value is a measure of the soil's density, it should be noted that there is no evidence of increasing density of the silty sand as depth increases. Sands are classified into relative density groups on the basis of this penetration test; NACA's soil, falling into the 10 to 30 blow group, is classed as "medium" sand by this test. (See page 294, reference 13.)

A study of the grain size makeup of the silty sand from the gradation charts is very revealing as it indicates the extent of homogenity of the silty sand over a wide area. To indicate the truth of the above statement, two graphs have been prepared. Figure 4-2 is a plot of representative samples from five widely scattered projects on the base. Figure 4-3 compares this scatter with the scatter obtained from the samples from one project.

The gradation curves shown on Figure 4-2 all show the same characteristics, that is, the complete absence of large

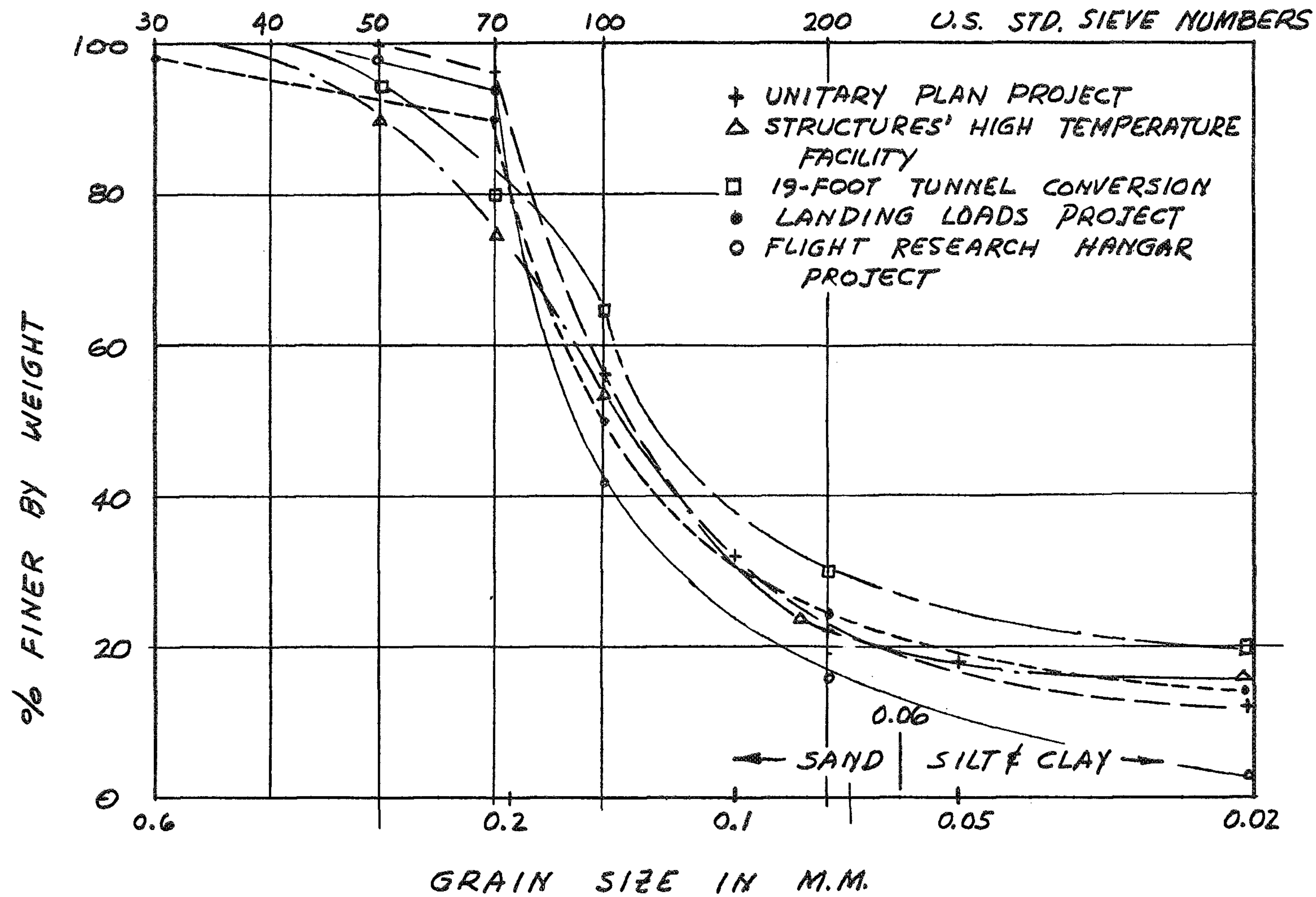
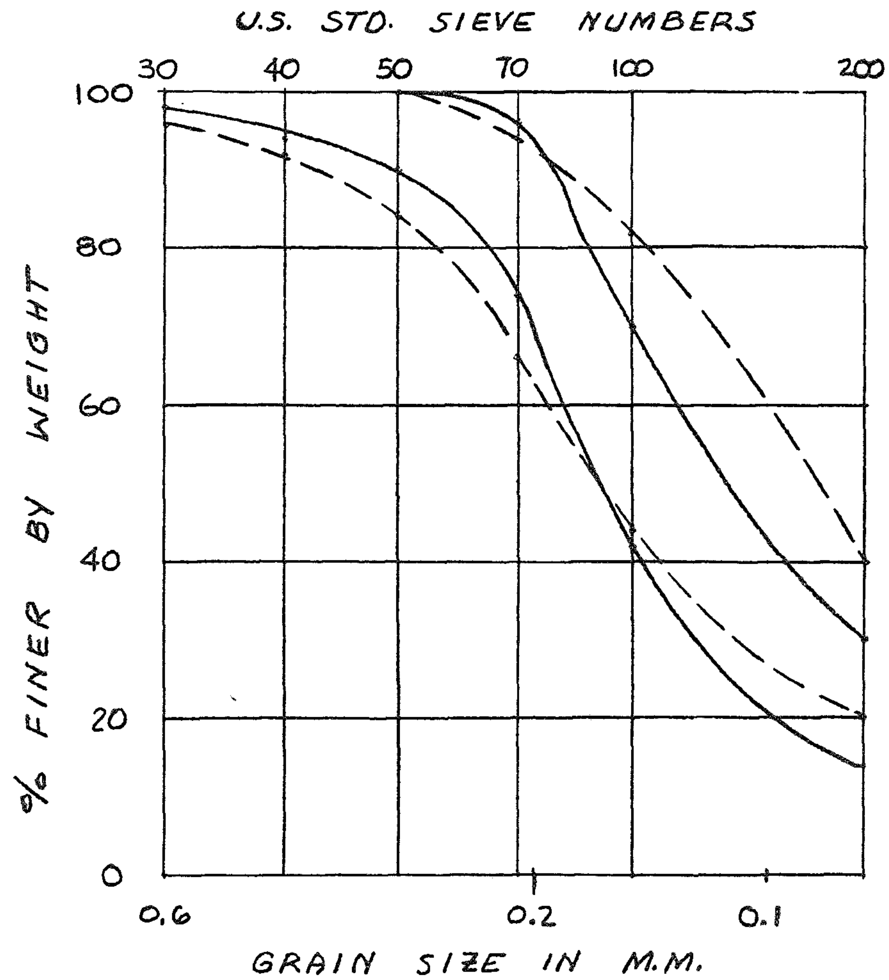


FIGURE 4-2

GRADATION CURVES OF SILTY-SAND STRATUM



REGION BETWEEN SOLID LINES REPRESENTS SCATTER BETWEEN TYPICAL GRADATIONS ON FIVE PROJECTS.

REGION BETWEEN DASHED LINES REPRESENTS THE SCATTER-BAND OBTAINED FROM THE SILTY-SAND SAMPLES ON A SINGLE PROJECT, THE 19-FOOT P.T. CONVERSION.

FIGURE 4-3

TYPICAL GRADATIONS ON FIVE PROJECTS PLOTTED AGAINST SCATTER BAND OBTAINED ON ONE PROJECT size grains, the sharp decline of the curve between sieves No. 70 and No. 100, and the linearity in the silt and clay region. In the matter of the large size grains, an inspection of the sieves, after running a gradation, reveals that practically all the soil sample failing to pass the No. 70 and larger sieves are shell fragments. The sharp decline signifies that the soil is not well graded; as a matter of fact, approximately 60 per cent of the soil falls within the narrow confines of 0.1 to 0.2 mm in grain diameter. The leveling-off of the curve in the silt and clay region means that these small grains are well graded for maximum density so that their varying sizes might enable them to fill the voids between the sand grains. Figure 4-3 is self-explanatory.

Probably the most important information to be gleaned from gradation curves is the soil's classification. The method of classifying soils is yet far from standardized; there are, perhaps, a dozen different methods now in use. One of the most popular, Figure 4-4, is the triangular diagram developed by the Public Roads Administration. The soil to be classified is approximately 80 per cent sand; thus, by the PRA chart, the soil could be termed either "sand" or "sandy-loam."

A very simple and sensible method is now gaining in use and is preferable. It consists merely in giving the

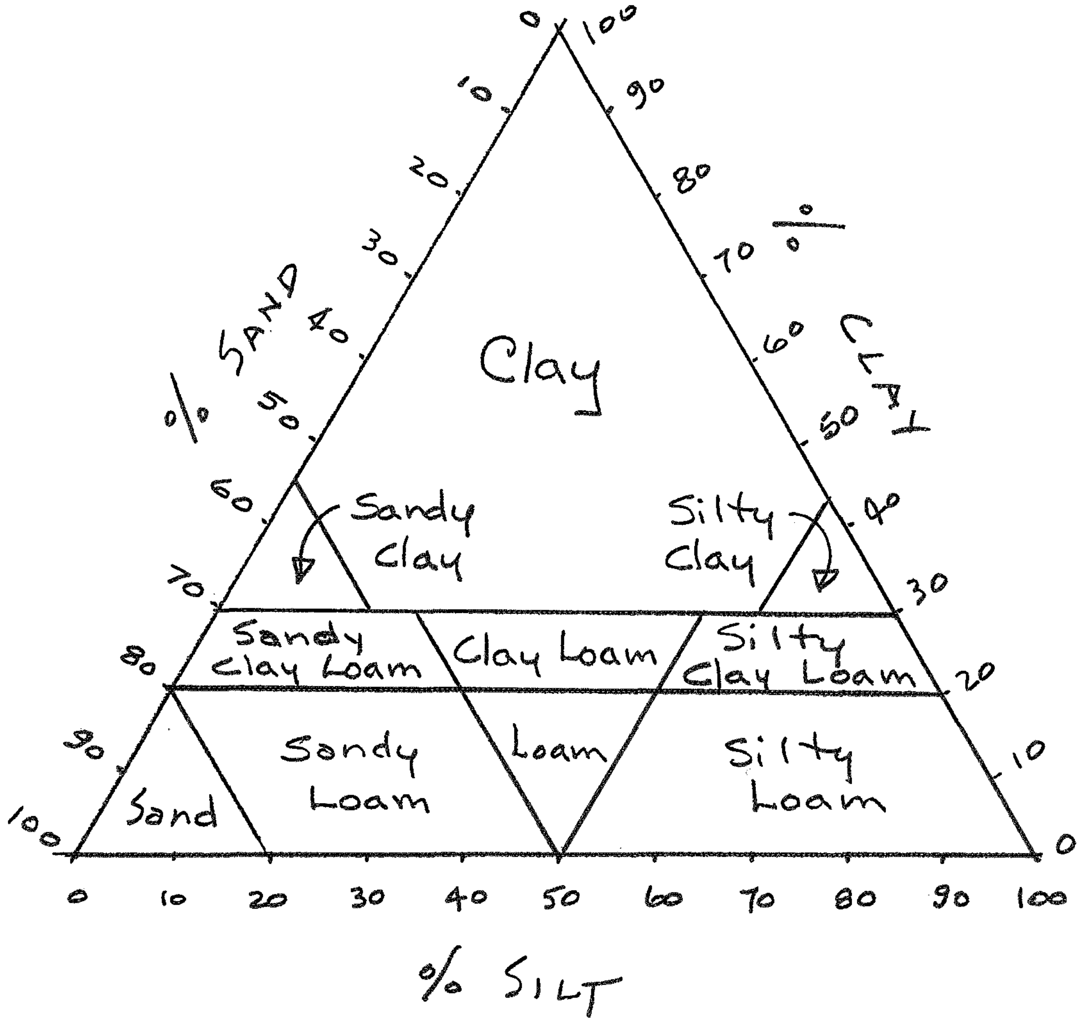


FIGURE 4-4

SOIL CLASSIFICATION CHART DEVELOPED BY THE PUBLIC ROADS ADMINISTRATION soil the name of its major constituent and modifying it with the adjective of its next constituent. The soil would, thus, be classified as "silty sand."

In classifying a soil, it is also useful to note its "effective size" (D_{10}) and its "uniformity coefficient" (U). These terms are a rough measure of the soil's permeability and grain size. D_{10} is the grain diameter on the gradation curve comparing to 10 per cent finer. In this case, $D_{10} \approx 0.05$ mm. Since U is equal to $D_{60} \div D_{10}$, $U \approx 3.4$.

Tests particularly useful for classifying clayey or silty soils are the liquid limit (L_w), the plastic limit (P_w), and the plasticity index (I_w). The Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi (reference 14) ran the above tests on the silty sand; they reported the approximate values of $L_w = 35$ and $I_w = 6$ as evidence of "low plasticity." The Haller Testing Laboratories (reference 4), after running similar tests, simply reported the silty sand as "nonplastic."

Other characteristics of the silty sand which have been determined by tests are tabulated below:

Natural water content = w = 23 to 33 per cent

Dry weight in place = γ_d = 85 to 95 p.c.f.

Natural void ratio = e = 75 to 100 per cent

To sum up, the soil under investigation is classified as a saturated silty sand (approximately 80 per cent sand) of negligible plasticity which extends to an undetermined depth without an appreciable change in density.

CHAPTER V

BEARING CAPACITY BASED ON LABORATORY TESTS

It is important to touch briefly upon the many difficulties hampering the soils engineer from making an exact solution of ultimate bearing capacity even with the use of laboratory tests to determine the soil's characteristics.

The problem of computing the ultimate bearing capacity of continuous footings has perhaps been more fully investigated theoretically than any other footing problem. The reason is twofold: (1) its importance since it is encountered frequently and (2) a transverse section can be taken which resolves the analysis into a plane strain problem. To solve this plane strain problem, all that one needs to assume is:

(1) Angle of internal friction = constant = ϕ

(2) Shearing resistance of the soil = s

(3) Cohesion value of the soil = constant = c

(4) Smooth base of the footing

(5) Unit weight of soil = γ

Approximate values of γ, c, and ϕ can be obtained from laboratory tri-axial shear tests. The value s can be found by the equation

$$s = c + p \tan \phi \tag{5.1}$$

where p is the normal pressure. The difficulty which arises is that the bases of footings are, without exception, rough rather than smooth. Thus, approximations have to be made in order to obtain a solution. An approximate solution has been obtained by Terzaghi by assuming that wedge I, Figure 5-1, acts with the footing. He then divides the subgrade into Rankine zones which can be solved independently for equilibrium. Zone I is the active Rankine zone, zones II are termed the zones of radial shear, and zones III the passive Rankine zones. The solution for a general shear failure is

$$Q_d = B\left(cN_c + \gamma D_f N_q + \frac{1}{2}\gamma B N_\gamma\right) \tag{5.2}$$

where

Q_d = ultimate load per foot on footing

B = width of footing

N_c, N_q, and N_γ = dimensionless factors, functions of ϕ

D_f = depth of foundation below grade level

Values of N_c, N_q, and N_γ are given in Figure 5-2 for various values of ϕ. There is, however, an implicit assumption made in this analysis; the assumption is that the footing remains horizontal. It is unfortunate, indeed, that all footings fail by tilting due to some nonhomogenity of the soil. Thus, the footing will fail at some load less

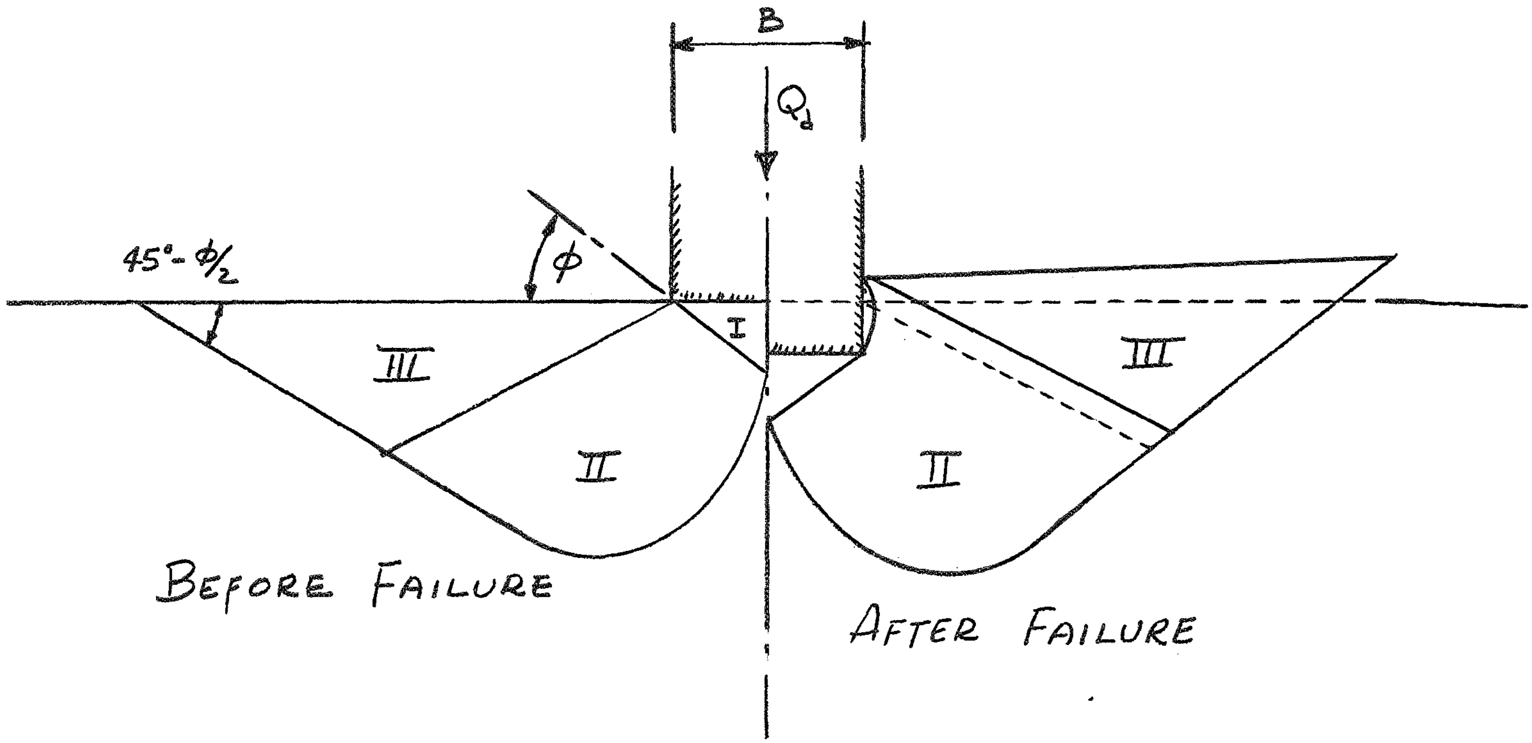


FIGURE 5-1

FAILURE OF A CONTINUOUS FOOTING WITH A ROUGH BASE

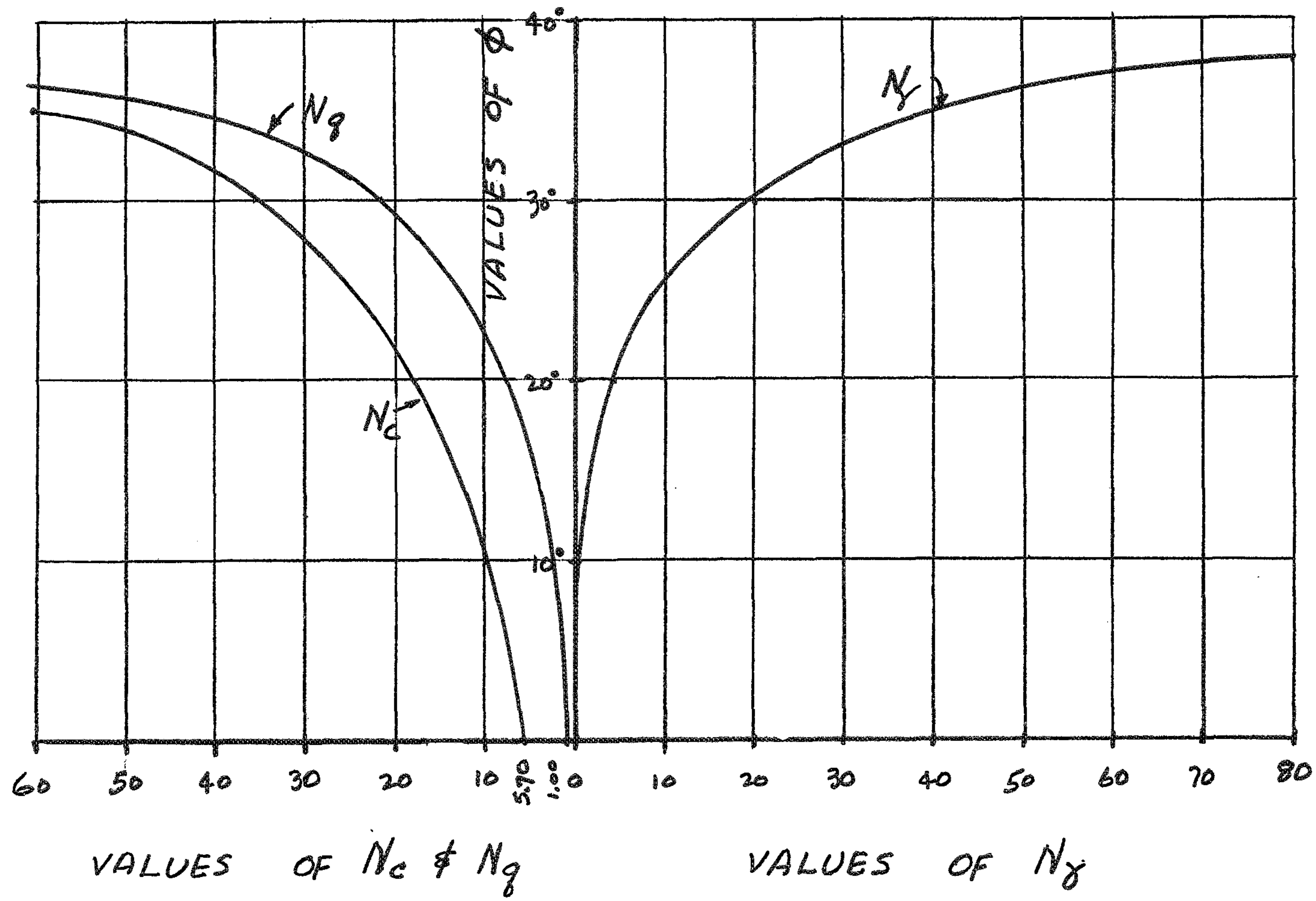


FIGURE 5-2

RELATION BETWEEN ϕ AND BEARING CAPACITY FACTORS than Q_d. But since there is no way to estimate the "tilt" factor, the above equation is probably the best available for estimating purposes.

Professor William S. Housel of the University of Michigan, feeling no doubt that the exactness has already been lost, presents a simpler mathematical solution (reference 6) by making the following simplifying assumptions:

(1) 45° triangular zones of sliding

(2) For silty sand classified in chapter IV,

$$s = \text{a constant} = c \; \boxed{\phi = 0}$$

Figure 5-3 illustrates his method; free bodies are taken of each triangle; by placing the sum of the vertical and horizontal forces equal to zero for each triangle, four equations are found which are sufficient to solve the four unknowns P_1, P_2, P_3, and Q_d. The solution obtained is

$$Q_d = 6Bc + B\gamma D_f \qquad (5.3)$$

Not even approximate theoretical formulas have yet been developed for square or circular footings. The best available formulas are semiempirical based on actual test data; they are:

(1) For circular footings of radius r

$$Q_{dr} = \pi r^2 \left(1.3cN_c + \gamma D_f N_q + 0.6\gamma r N_\gamma\right) \qquad (5.4)$$

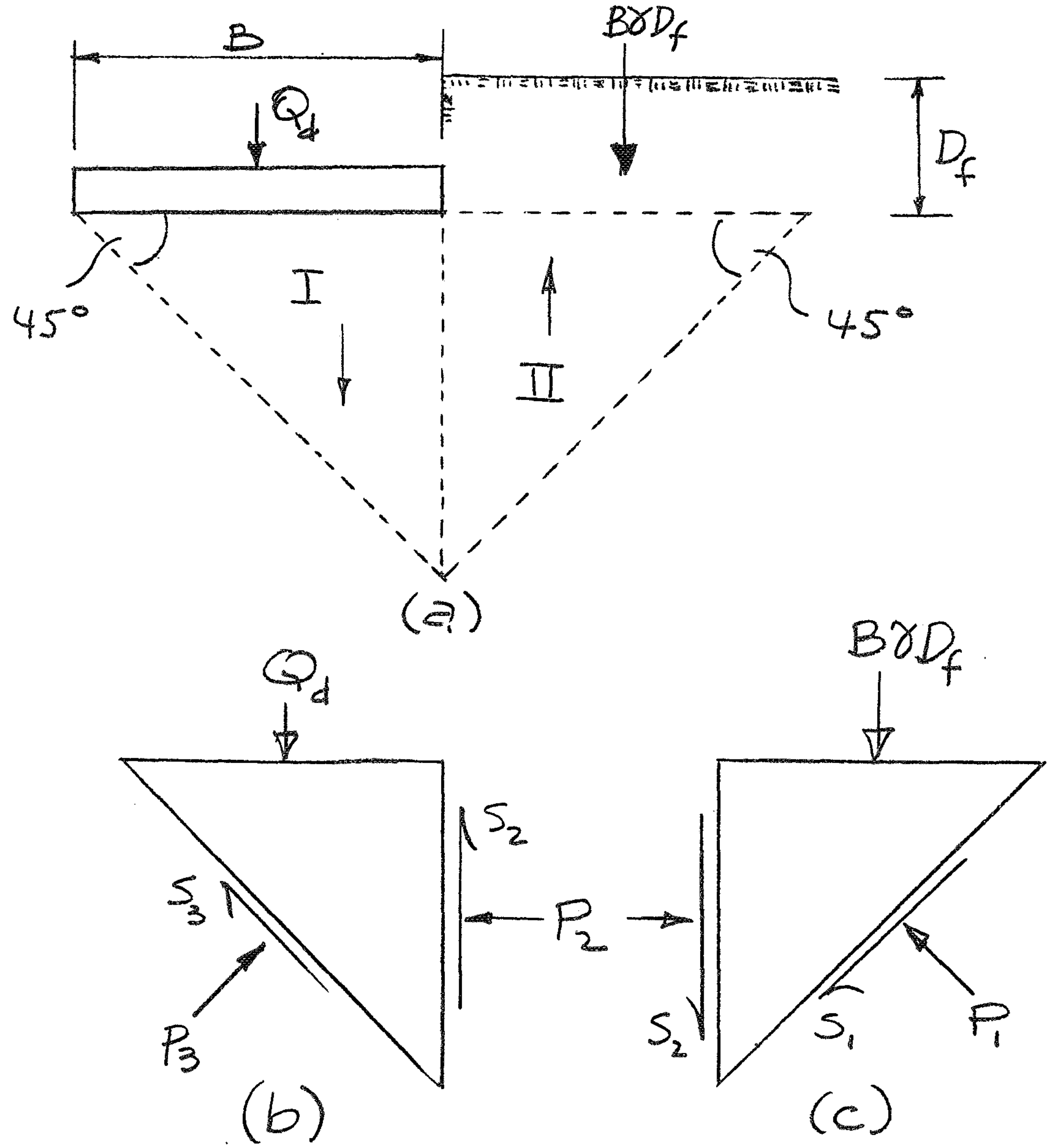


FIGURE 5-3

HOUSEL'S METHOD OF SOLVING FOR CRITICAL LOAD ON CONTINUOUS FOOTING

(2) For square footings of side b

$$Q_{db} = b^2\left(1.3cN_c + \gamma D_f N_q + 0.4\gamma b N_\gamma\right) \qquad (5.5)$$

The values of N_c, N_q, and N_γ are given in Figure 5-2.

Equations (5.2), (5.3), and (5.4) will now be applied to some sample problems in order to check their agreement and to obtain some idea as to the bearing value of the silty sand.

It is first necessary to estimate ϕ and c from tri-axial shear tests. Figures 5-4 through 5-6 are included to present results of all tri-axial shear tests which have been run on the silty sand. Figure 5-7 is a composite plot of the average results from four projects. For substitution in the equations previously obtained, the average values of c = 0.33 ton per square foot and $\phi = 24^\circ$ will be used since comparison will be made with actual field tests.

The calculated ultimate load per foot on a 3'-0" wide footing located 3'-0" below grade by equations (5.2) and (5.3) is:

(1) By equation (5.2) Q_d = 30 tons per foot

q_d = 10 tons per square foot

(2) By equation (5.3) $Q_d = 6\frac{1}{2}$ tons per foot

q_d = 2 tons per square foot

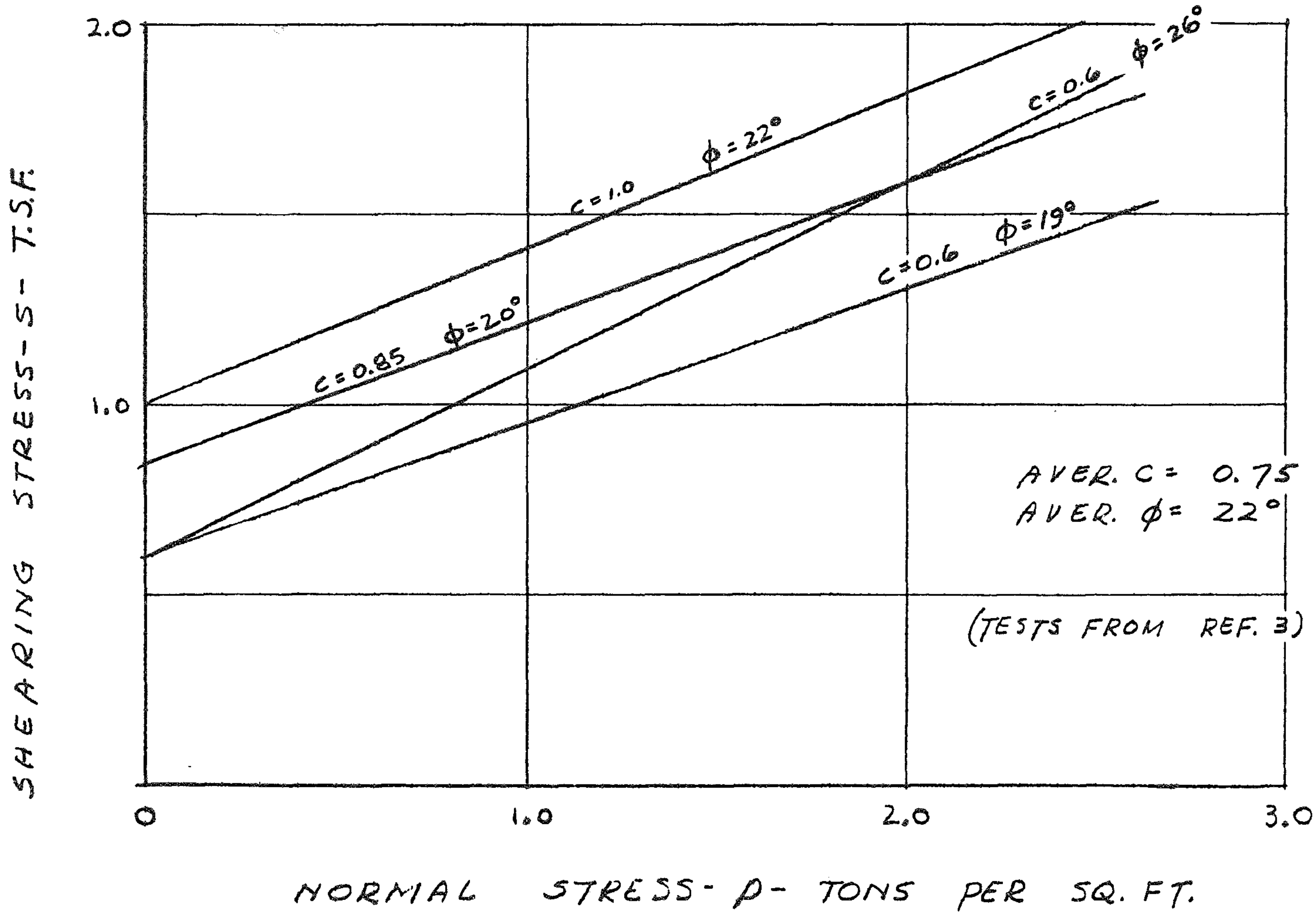


FIGURE 5-4

TRI-AXIAL SHEAR TEST RESULTS, GREY SILTY SAND,
LANDING LOADS PROJECT

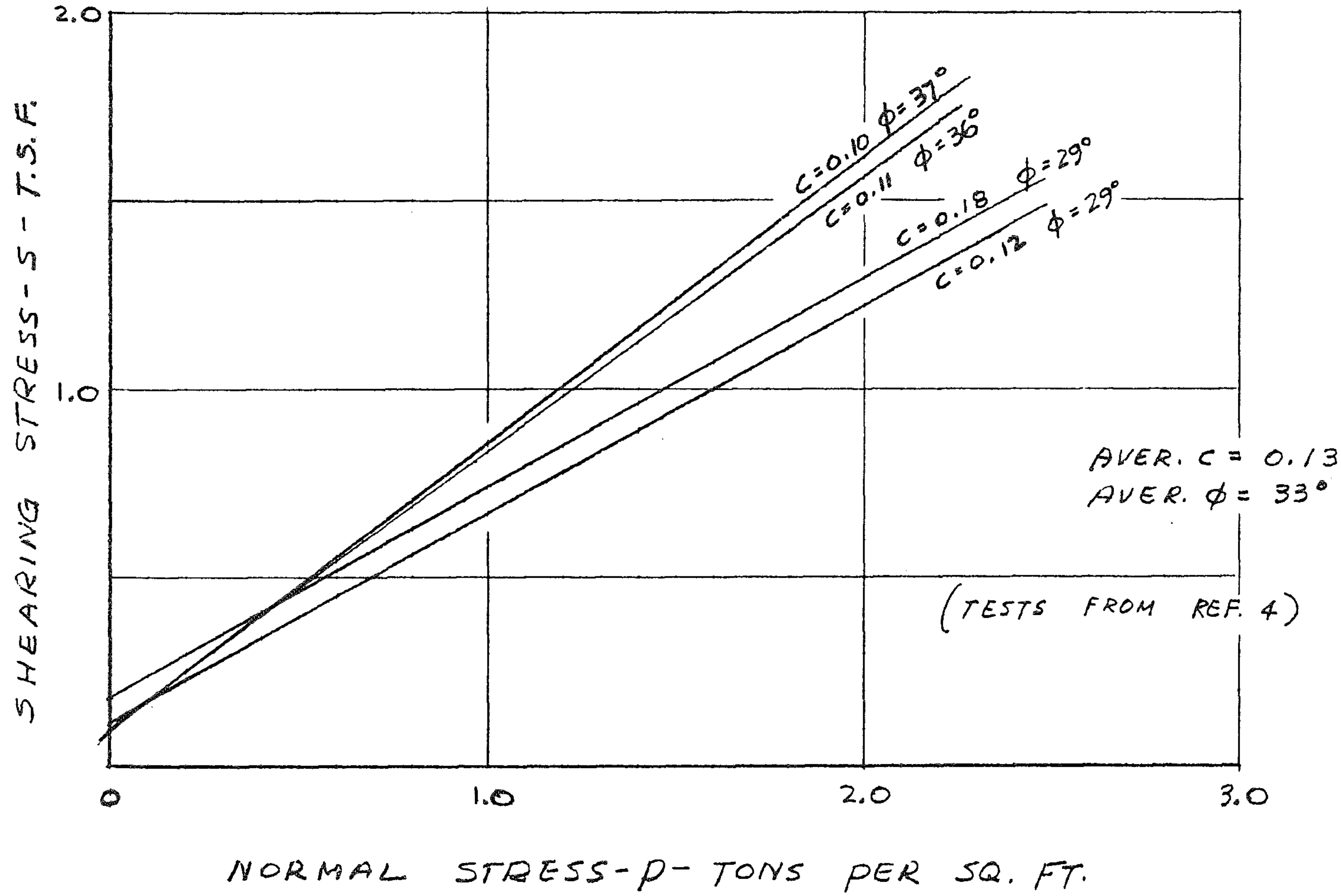


FIGURE 5-5

TRI-AXIAL SHEAR TEST RESULTS, GREY SILTY SAND,
19-FOOT PRESSURE TUNNEL CONVERSION

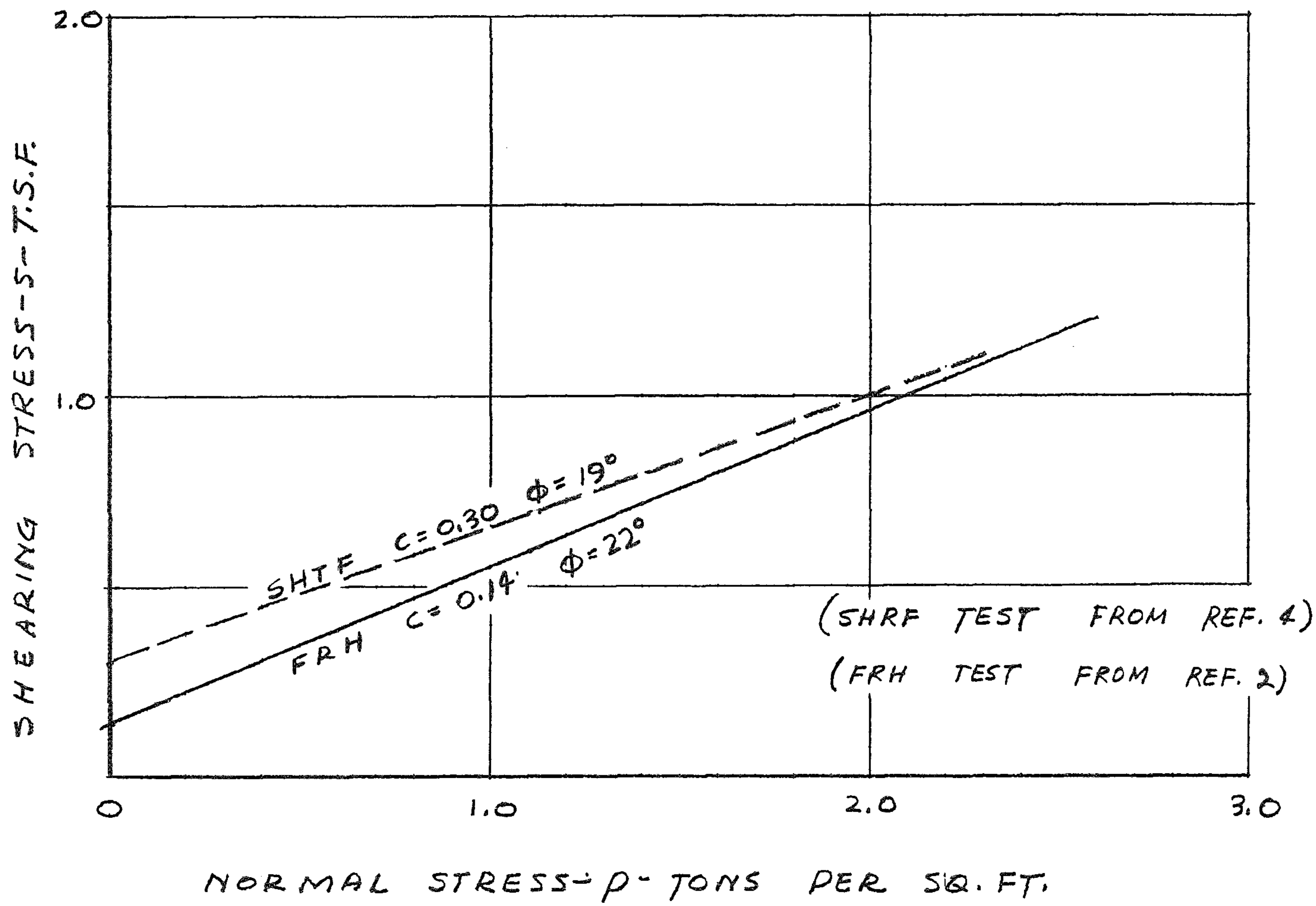


FIGURE 5-6

TRI-AXIAL SHEAR TEST RESULTS, GREY SILTY SAND, FLIGHT RESEARCH HANGAR, STRUCTURES HIGH TEMPERATURE FACILITY

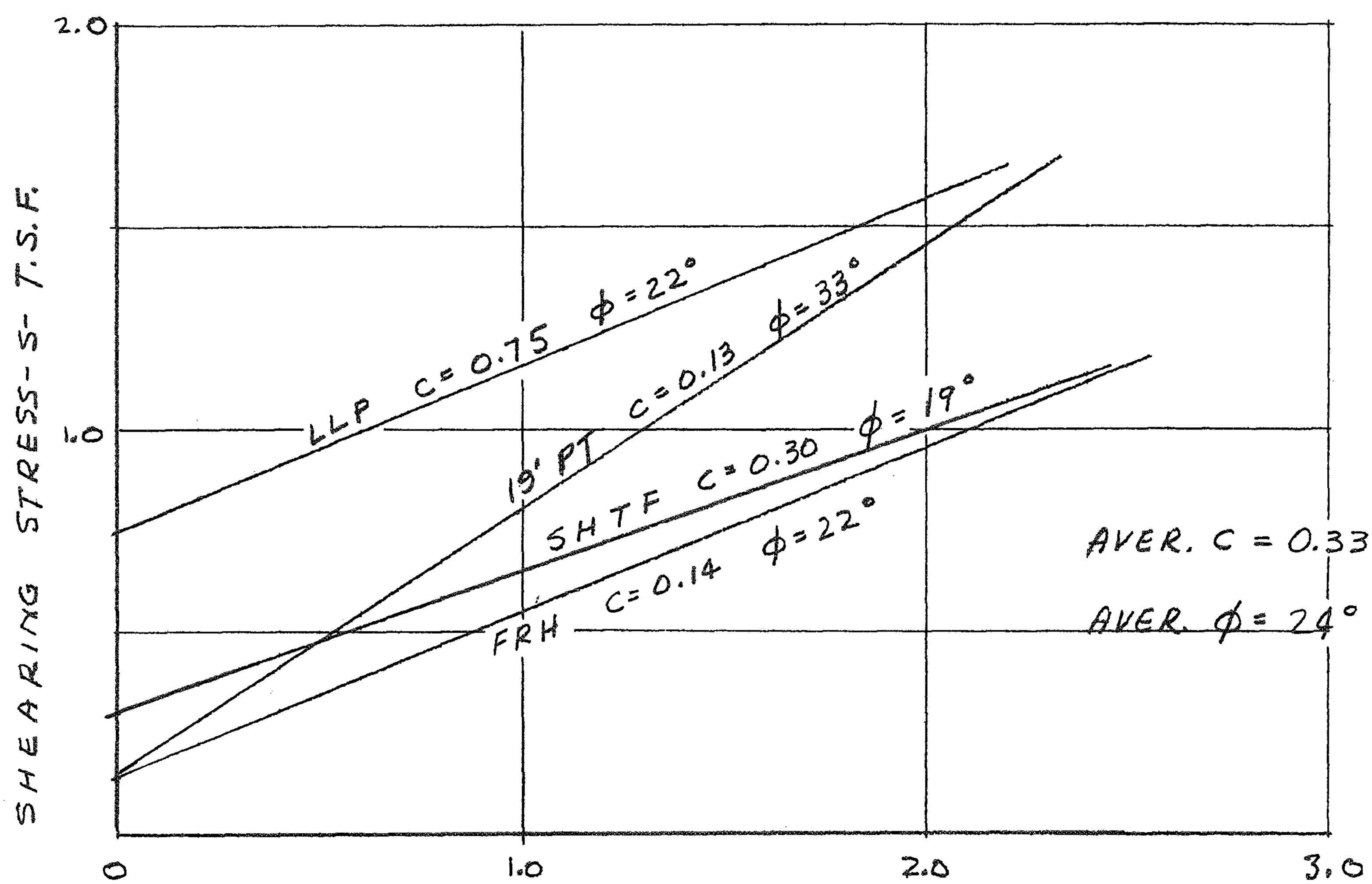


FIGURE 5-7

AVERAGE TRI-AXIAL SHEAR TEST RESULTS FROM FOUR PROJECTS,
GREY SILTY SAND

The great variance obtained in ultimate loads for a footing by use of different equations is evidence that a formula alone will not solve the soils engineer's problems. The formula is only one tool which the soils engineer uses with his other tools of common sense and experience in deciding upon an allowable bearing value.

Later in this thesis, test results on a 1-foot plate and on a 2-foot square plate will be presented; for comparison purposes, theoretical failure loads by equation (5.5) have been calculated. The results are

(1) 1-foot square plate (no surcharge)

$$Q_{db} = 11 \text{ tons}$$

(2) 2-foot square plate (no surcharge)

$$Q_{db} = 44 \text{ tons}$$

Piles carry their ultimate load Q by a combination of point resistance (Q_p) and skin friction (Q_f), In some cases, one or the other is negligible and the piles are termed either "point-hearing" or "friction" piles. Thus, the following equation is easily derived:

$$Q = Q_p + Q_f$$

$$Q = Q_p + 2\pi r D_f f_s \qquad (5.6)$$

Q_p can be calculated by equation (5.4) but since there is no theoretical analysis yet developed to obtain f_s

from laboratory tests, Q cannot be calculated. Load tests are the only way that Q can be obtained accurately. However, for design purposes when load tests are not available, the recommended procedure is to estimate f_s from tri-axial shear test results or from empirical values for soil types as tabulated in standard soil texts. Figure 5-8 is a plot of estimated ultimate pile capacities for various lengths based on the following assumptions:

(1) $r = 6$ inches

(2) $f_s = c = 666$ pounds per square foot

(3) $\phi = 24°$

(4) Q_p obtained from equation (5.4)

(5) γ' = submerged unit weight = 35 p.c.f.

An inspection of Figure 5-8 reveals that piles driven in the silty sand are primarily "friction" piles. For a 40-foot pile, the division is approximately 75 per cent friction and 25 per cent point resistance.

Using the results from equations (5.2) and (5.6) with a recommended safety factor of 2-1/2 (it is felt that the assumption that $\phi = 0$ in equation (5.3) is ultra-conservative), the following approximate allowable bearing values are obtained:

(1) For spread footings

$q_a = 4$ tons per square foot

(2) For a 40-foot pile

$Q_a = 23$ tons

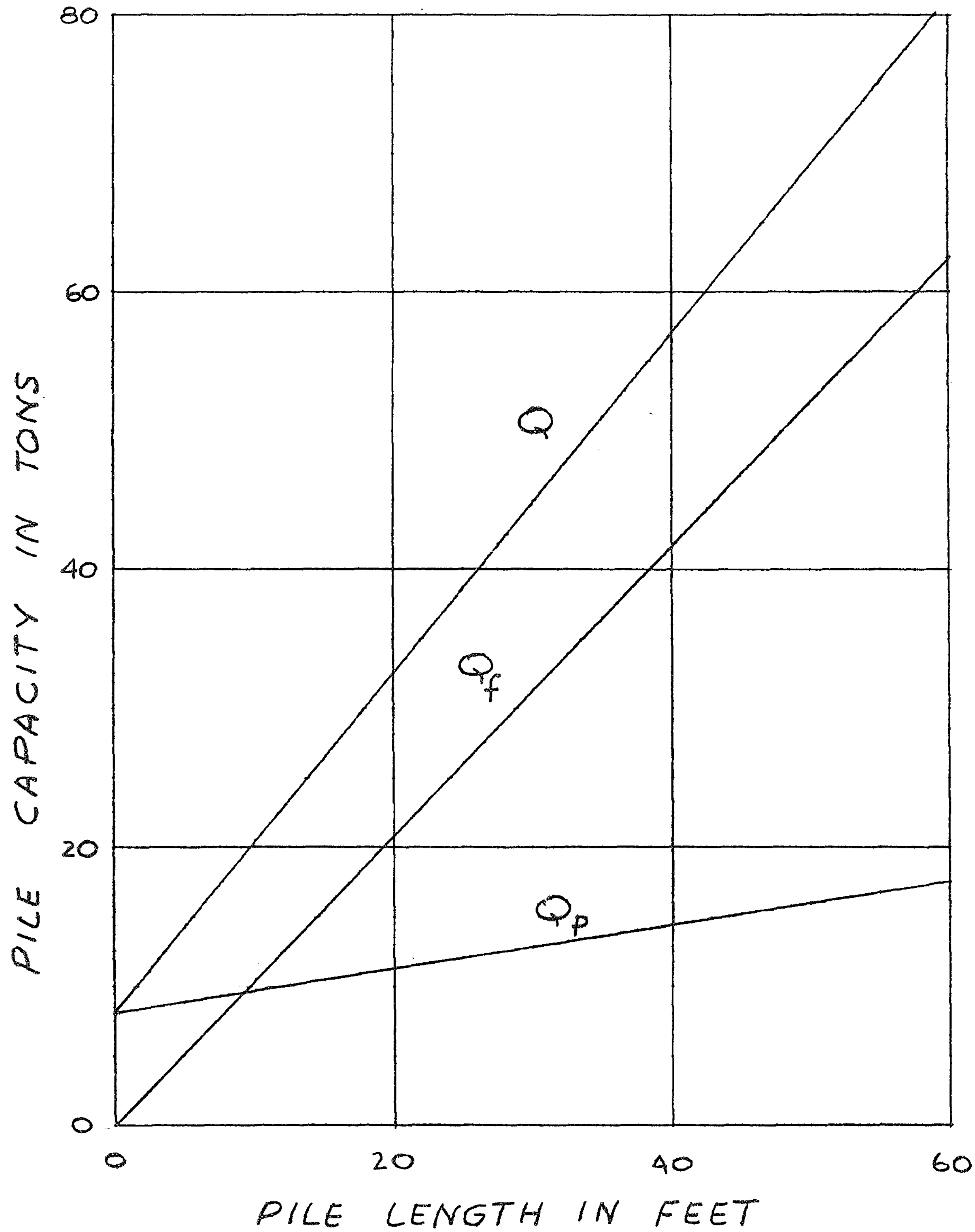


FIGURE 5-8

THEORETICAL ULTIMATE PILE LOAD FOR VARIOUS LENGTH PILES

CHAPTER VI

ESTIMATED SETTLEMENT BY LABORATORY CONSOLIDATION TESTS

It is acknowledged that an exact prediction of expected settlement cannot be made except in exceptional cases. In most cases, the subsoil is not uniform and, also, there is always some disturbance of the soil in taking an "undisturbed" sample for laboratory testing. It is, however, necessary that the designer be able to estimate approximately the expected settlement so that he will be in a position to recognize what factors are negligible and which are liable to cause trouble, necessitating close inspection during the construction period.

The theoretical solution is made up of the following components: (1) the determination of the vertical pressure at various depths under the load by means of Boussinesq's stress equations, (2) the laboratory consolidation tests on representative samples, and (3) the determination of the settlement from use of the actual vertical pressure in conjunction with the laboratory test results.

To outline the procedure fully, the pile cap shown on Figure 6-1 will be investigated for its estimated settlement.

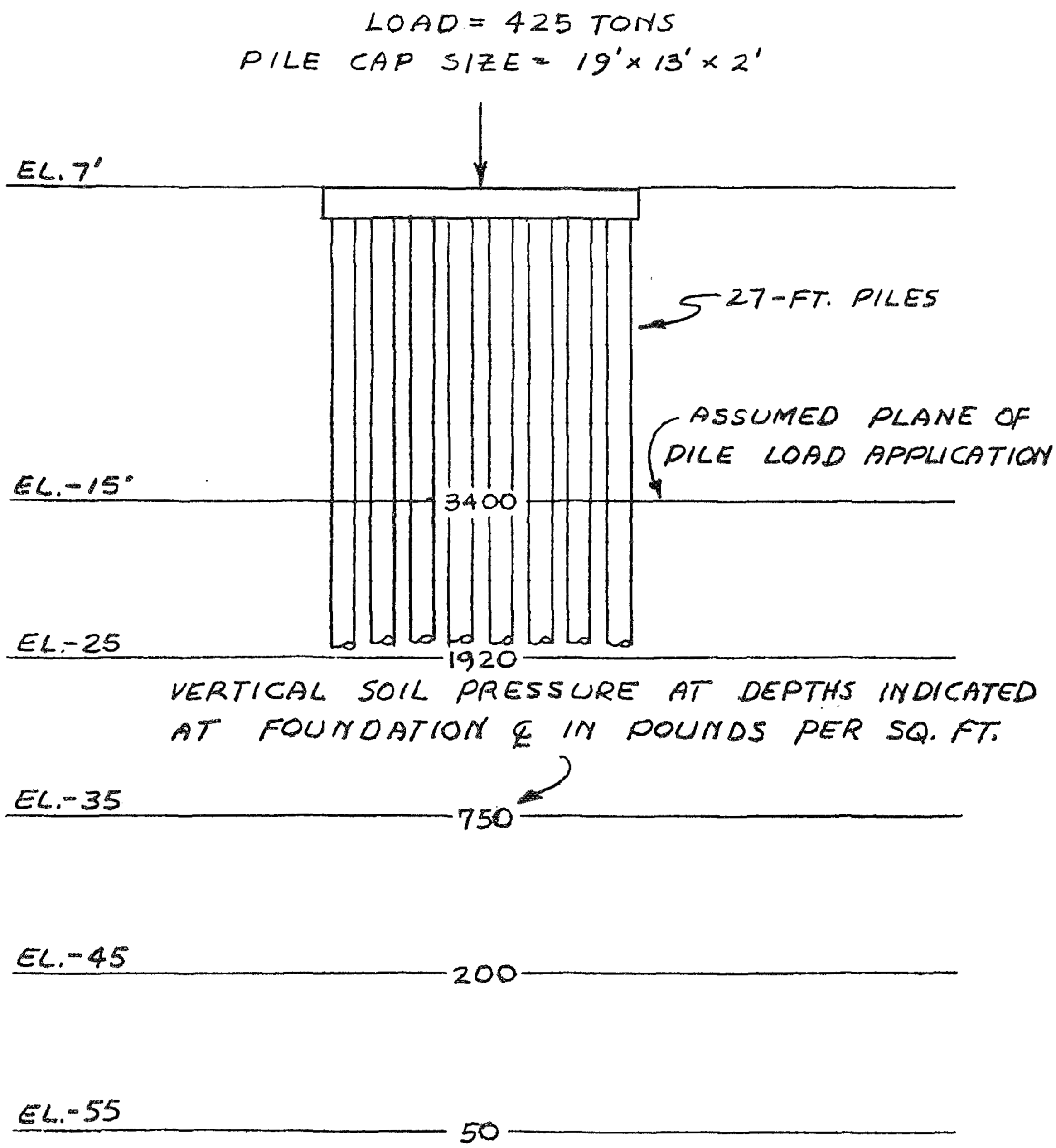


FIGURE 6-1

VERTICAL PRESSURES UNDER A LOADED PILE FOUNDATION

The first step is to assume a plane of the pile load application. It is usually assumed to be two-thirds the length of the pile from the top but this is only a rough approximation since the location of said plane depends also on the characteristics of the soil and the magnitude of the load.

For the case under investigation, the plane of pile load application is assumed to be at el. -15'. The vertical pressure (p_v) at this elevation is thus:

$$p_v = \frac{850 \text{ kips}}{19' \times 13'} = 3.40 \text{ k.s.f.}$$

Boussinesq developed the following equation for the determination of vertical pressure at any point, either directly below or laterally displaced beneath the load:

$$p_v = \frac{3L}{2\pi z^2}\left[\frac{1}{1 + \left(\frac{r}{z}\right)^2}\right]^{5/2} \qquad (6.1)$$

where

z = vertical distance from load application

r = horizontal distance from c.g. of load

L = load

Equation (6.1) is easily solved by use of an influence chart developed by N. M. Newmark. His chart is shown on Figure 6-2 for an influence value of 0.005. The chart is used in the following manner: the plan of the loaded area is drawn to a scale such that z is equal to the length of line AB, the plan is then located such that the point r is at the center of the chart, the number of blocks or "influence areas" enclosed by the plan are then counted, and p_v is equal to the product of 0.005, the number of influence areas, and the p_v at $z = 0$.

For $z = 10$ and $r = 0$, the number of blocks enclosed by the pile cap 19' × 13' is 113 so that

$$p_v = 113 \times 0.005 \times 3.40 = 1.92 \text{ k.s.f.}$$

The vertical pressures thus computed are tabulated on Figure 6-1.

The next step is to determine the coefficient of volume compressibility (m_v). This is accomplished by use of the laboratory pressure-void ratio curve (Fig. 6-3), the calculated vertical pressures, and the following equation:

$$m_v = \frac{e_o - e}{\Delta p_v(1 + e_o)} \qquad (6.2)$$

e_o = void ratio at original pressure

e = void ratio at loaded pressure

Δp_v = loaded pressure - original pressure

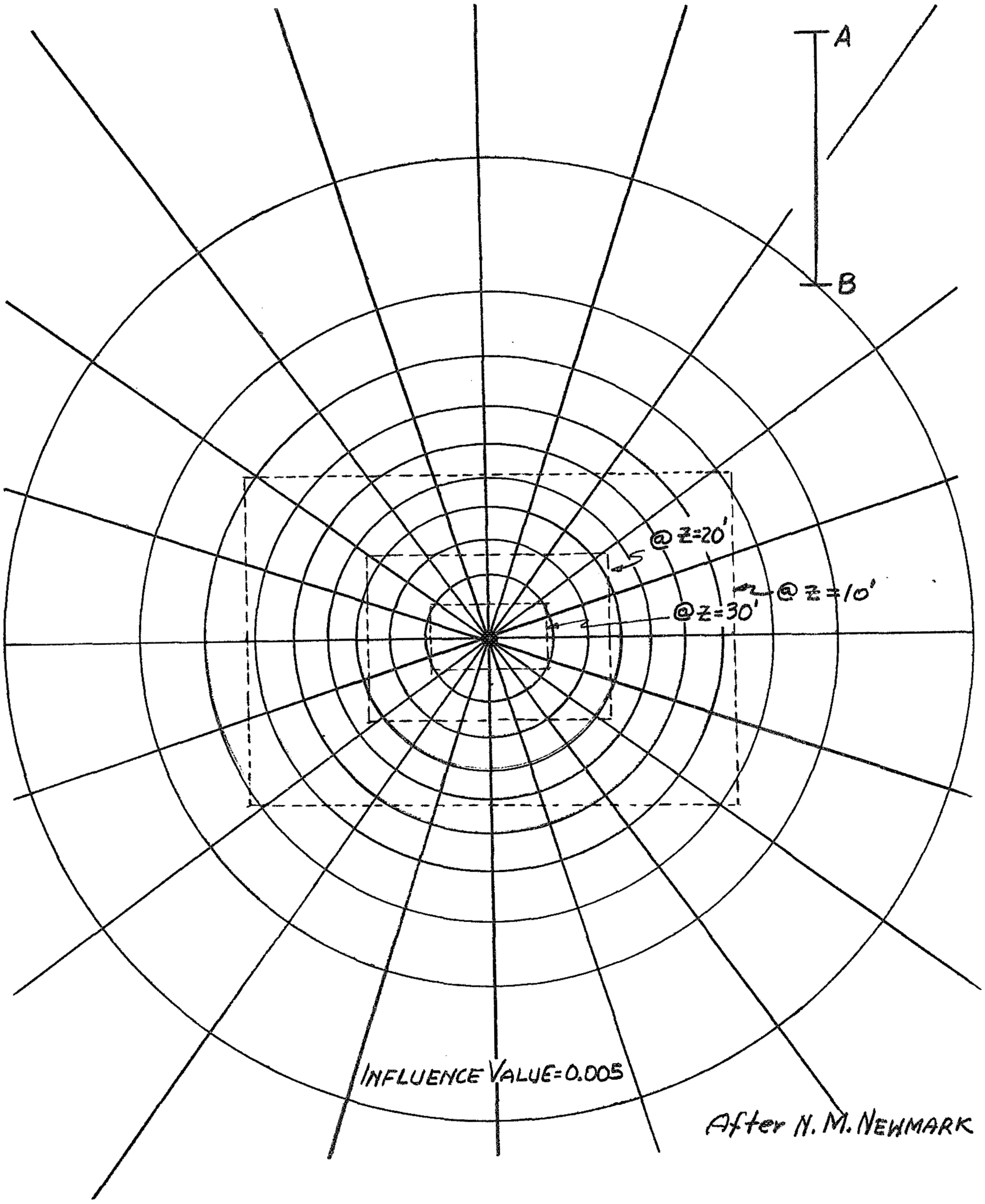


FIGURE 6-2

INFLUENCE CHART FOR VERTICAL PRESSURE

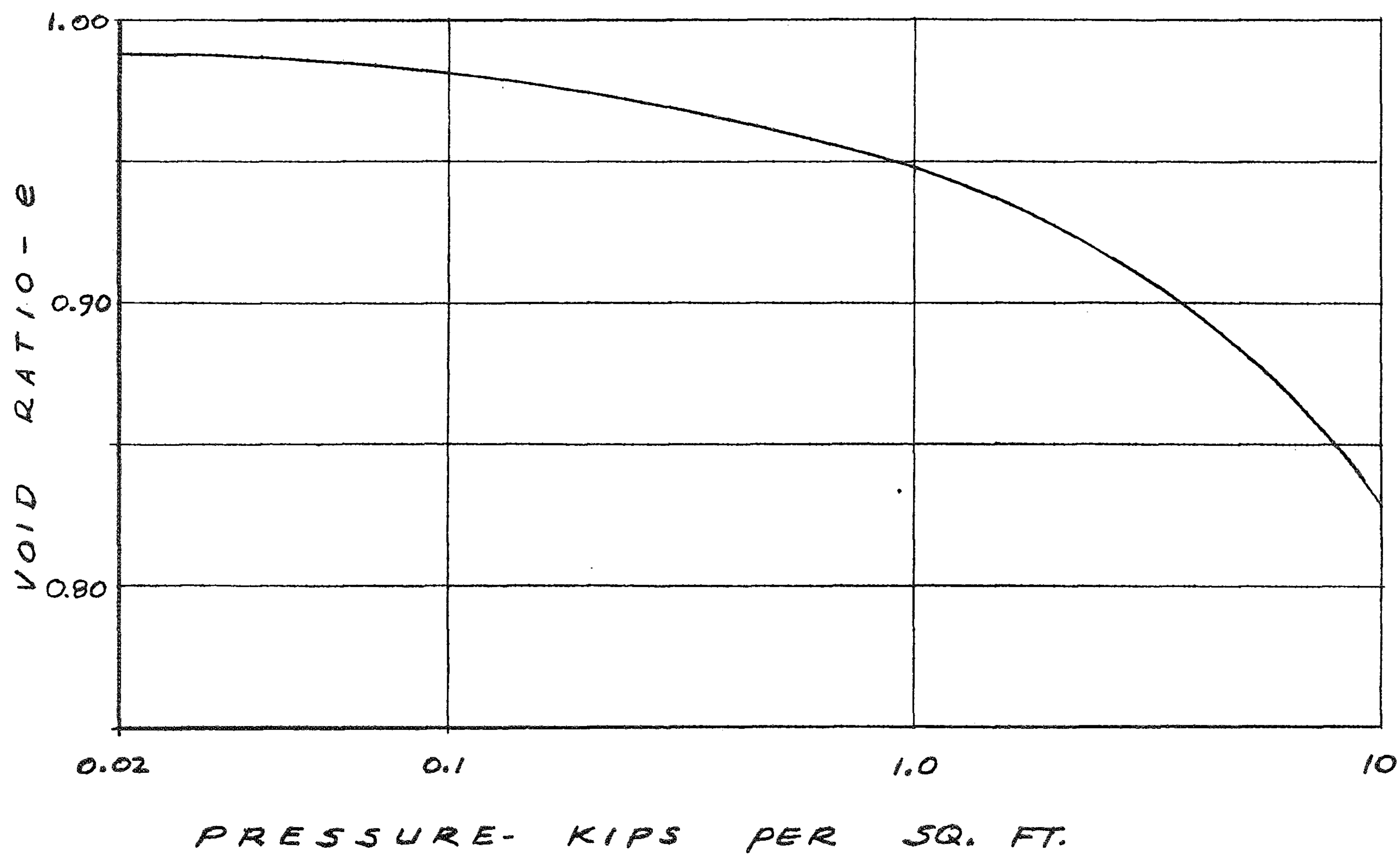


FIGURE 6-3

PRESSURE-VOID RATIO CURVE

For example, m_v will be calculated for a depth of 20 feet below grade. In Figure 6-4 (a), the overburden and effective pressures are plotted against depth below grade. This graph indicates an overburden pressure of 1.0 k.s.f. and a total pressure of 4.5 k.s.f. at 20 feet below grade.

The void ratios for the above pressures are taken from Figure 6-3. The following results are thus obtained for 20 feet below grade:

$$p_o = 1.00 \text{ k.s.f.} \qquad e_o = 0.945$$
$$p = 4.50 \text{ k.s.f.} \qquad e = 0.895$$

Therefore,

$$m_v = \frac{0.050}{3.5 \times 1.945} = 0.0073 \text{ s.f./kips}$$

The plot of m_v against depth below grade is obtained in this manner and is shown on Figure 6-4 (b).

The total theoretical settlement is given by the following formula:

$$\delta_T = \int_0^z m_v p_v \, dz \qquad (6.3)$$

This equation is most easily solved by graphical integration illustrated on Figure 6-4 (c). The product of m_v and p_v is plotted versus depth and inclosed area is equal to the settlement; it should be noted that this is true dimensionally. For the illustrated case:

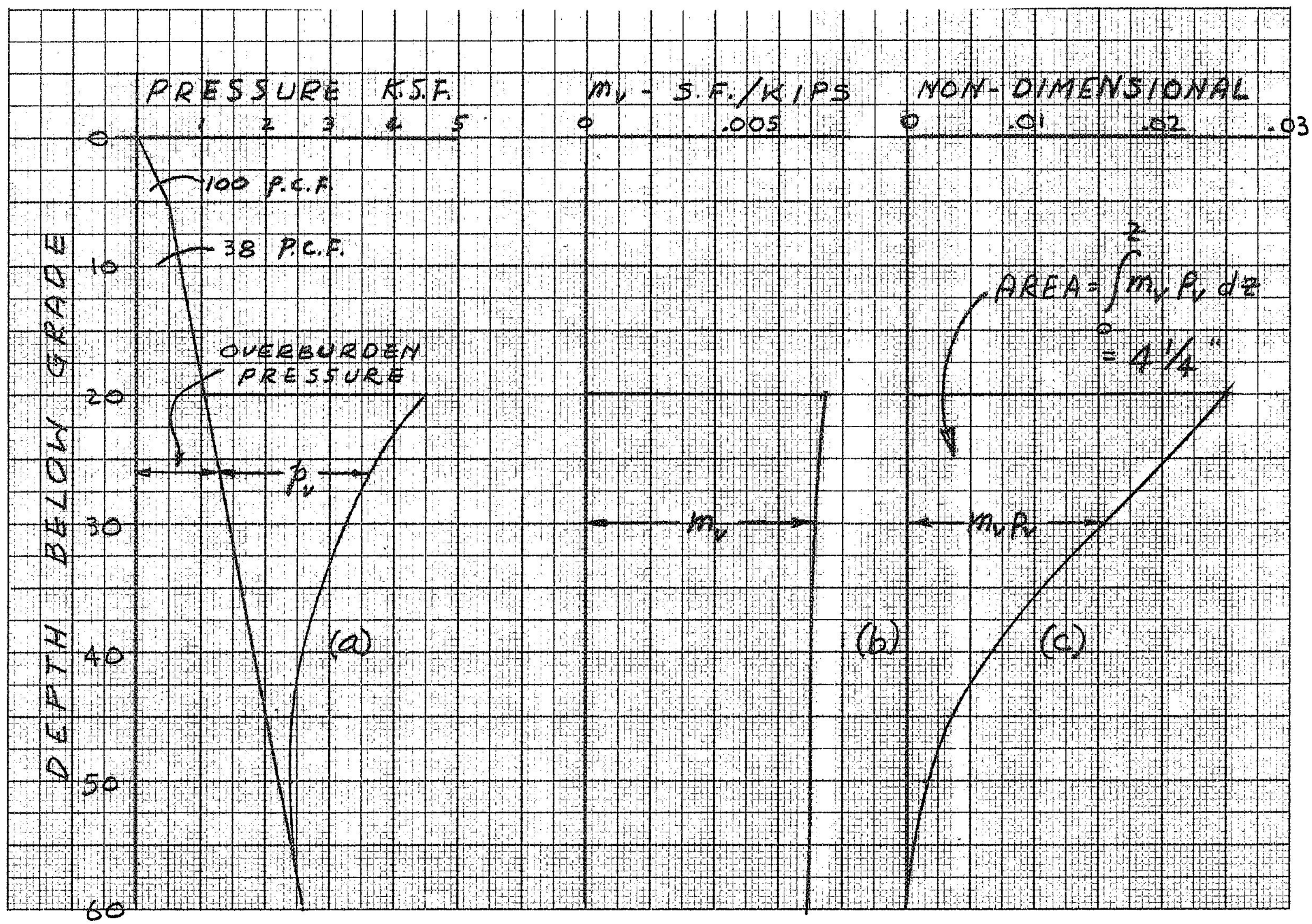


FIGURE 6-4

GRAPHICAL METHOD FOR COMPUTING SETTLEMENT-PILE FOUNDATION

$$\delta_T = 0.350' = 4.2''$$

Thus, for the case investigated, the estimated settlement would be approximately 4-1/4 inches.

The only difference in computing the estimated settlement for a spread footing is that the plane of the applied load is at the bottom of the footing. A sample solution for a spread footing is shown on Figures 6-5 and 6-6. A compressor foundation was chosen for illustrative purposes; theoretical calculations predict a settlement of approximately 1 inch.

The time rate of consolidation is especially important to the engineer since settlement during construction is usually not damageable. The designer is mainly concerned with the amount of settlement after the building lines have been finally set, the walls plastered, etc.

Neglecting secondary consolidation, consolidation theoretically proceeds according to the following formula:

$$t = T_v \frac{H^2}{c_v} \tag{6.4}$$

where

t = time

T_v = time factor for percentage of consolidation in time t

H = one-half thickness of the consolidating layer

c_v = coefficient of consolidation

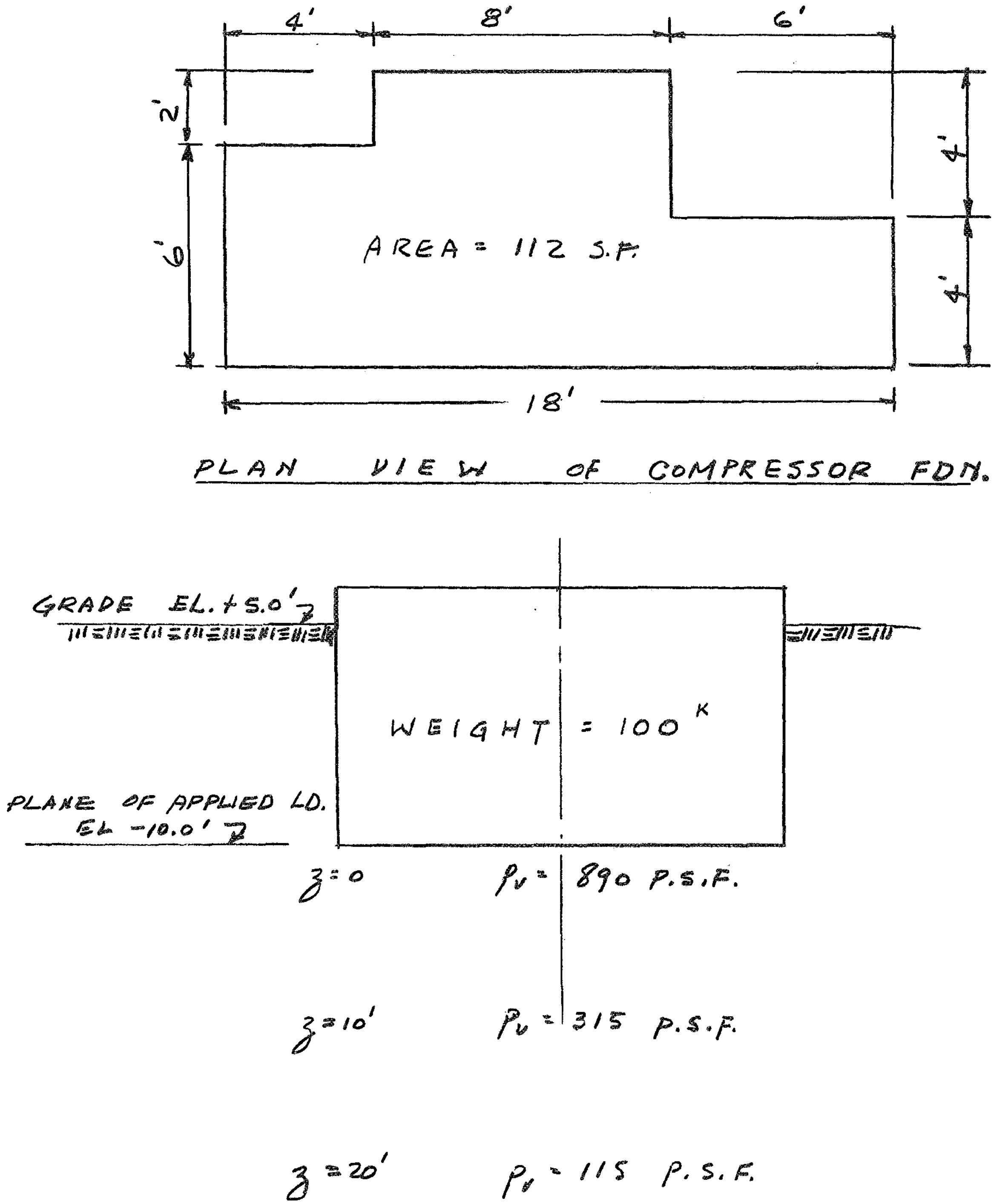


FIGURE 6-5

VERTICAL PRESSURES UNDER A SPREAD FOOTING

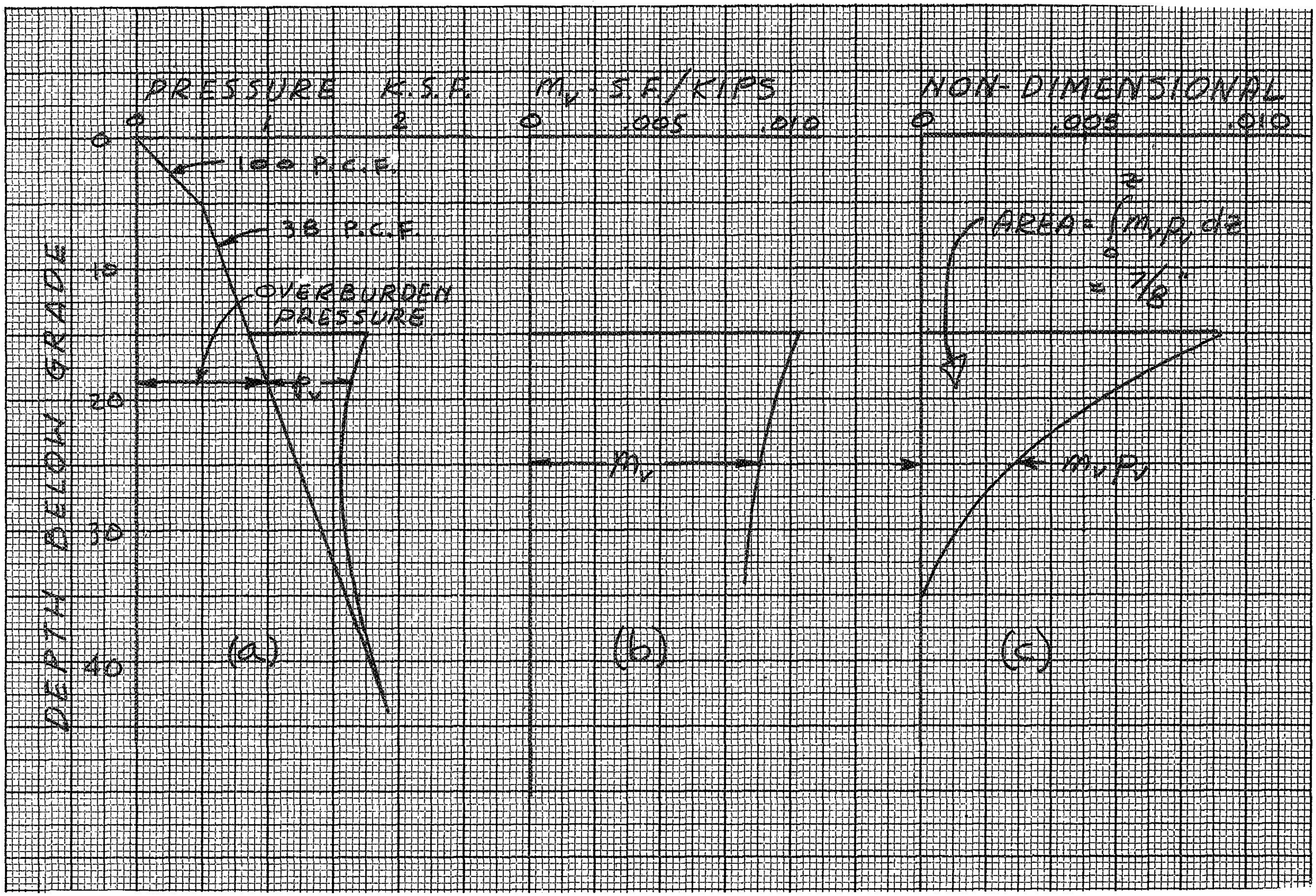


FIGURE 6-6

GRAPHICAL METHOD FOR COMPUTING SETTLEMENT-SPREAD FOOTING

T_v is a function of the per cent of consolidation (U%). Figure 6-7, from reference 11, is a graph showing how T_v varies with respect to U%. It is usually assumed in the case of an infinite, or very large, compressible stratum that H is equal to the width of the foundation since the consolidating pressure becomes negligible at a depth approximately equal to twice the width. The coefficient of consolidation can be determined only by laboratory test since it is a function of permeability; for the grey silty-sand stratum, Mr. Edward S. Barber (reference 1) found an approximate value of $c_v = 2$ square feet per day.

The procedure for obtaining a theoretical graph of settlement versus time has thus been established. The total settlement is calculated from equation (6.3) and is made equal to 100 per cent consolidation. The time for other percentages of settlement to take place are then computed by equation (6.4) with the aid of Figure 6-7.

Graphs of time-settlement relations for both the illustrative problems are presented in Figures 6-8 and 6-9.

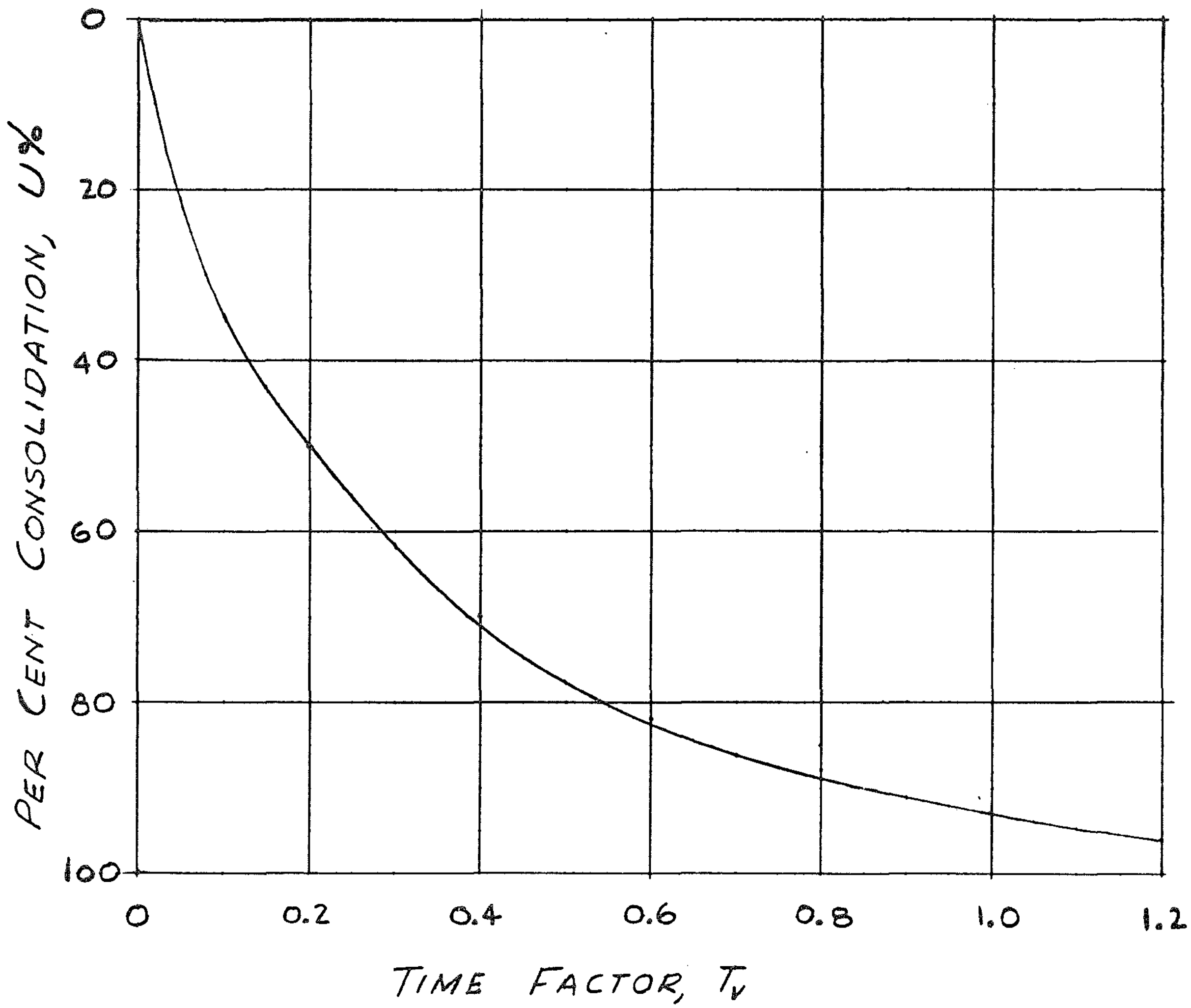


FIGURE 6-7

TIME RATE OF CONSOLIDATION

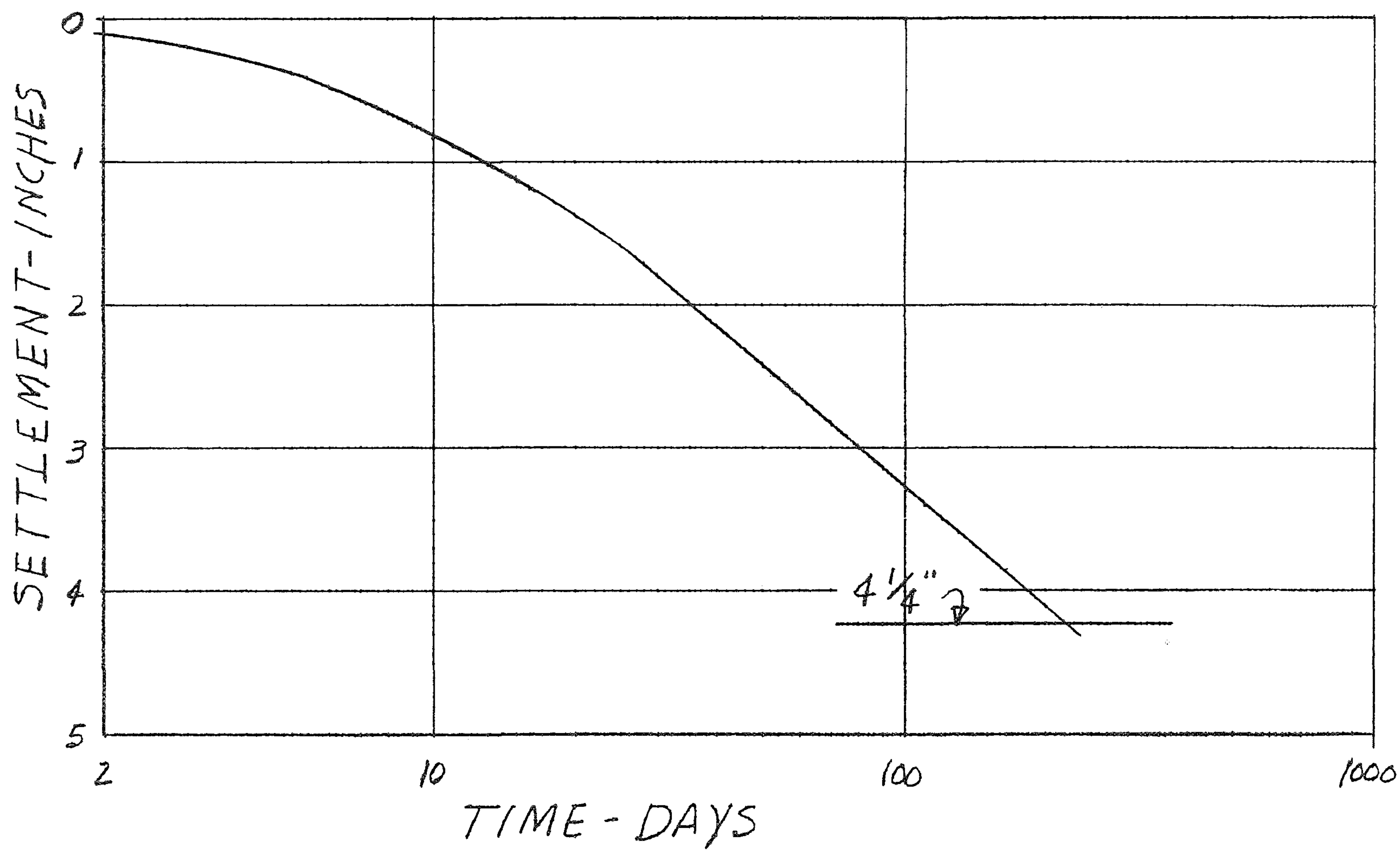


FIGURE 6-8

SETTLEMENT OF PILE FOUNDATION--THEORETICAL

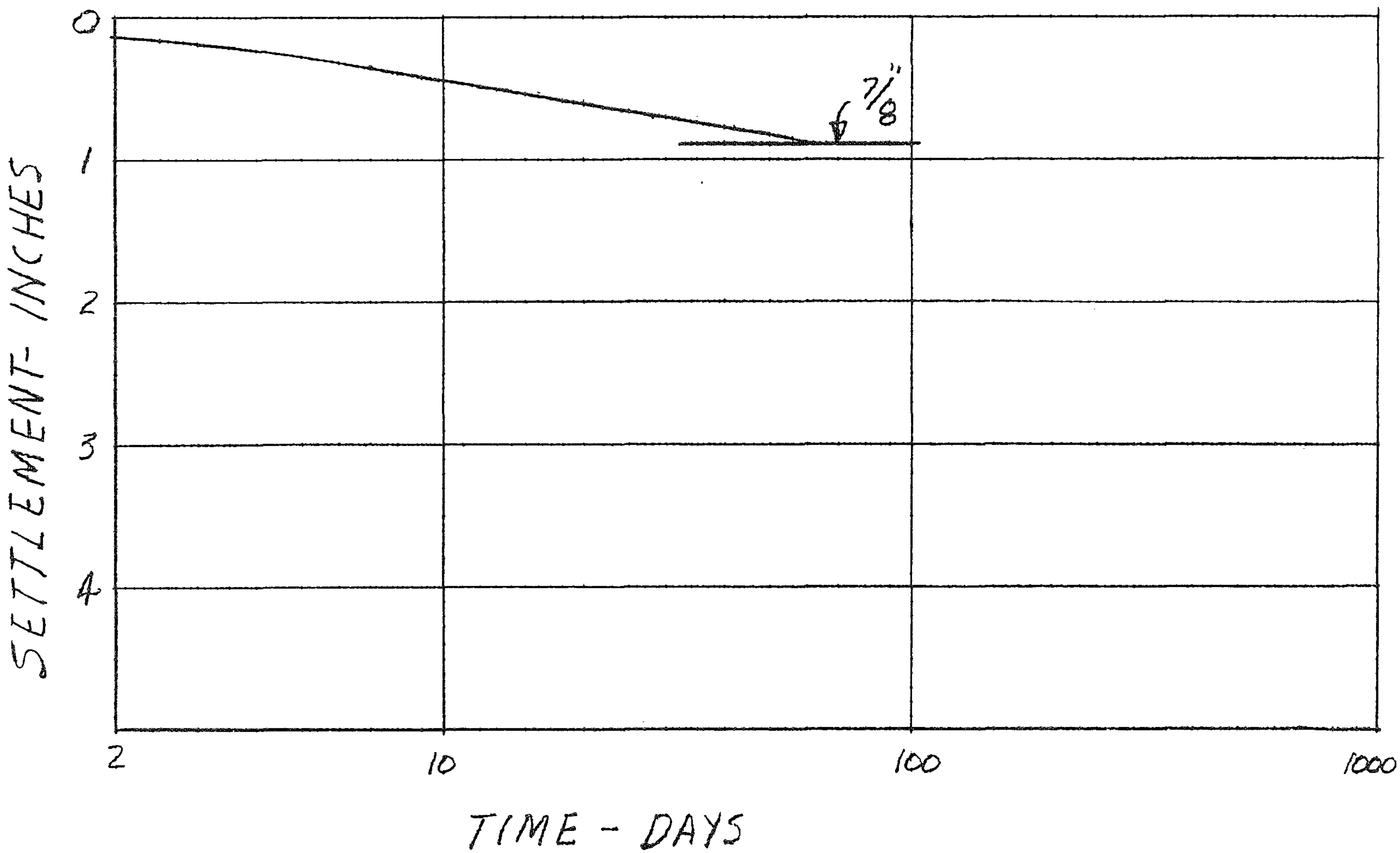


FIGURE 6-9

SETTLEMENT OF COMPRESSOR FOUNDATION--THEORETICAL

CHAPTER VII

BEARING CAPACITY AS REVEALED BY FIELD TESTS

Field load tests are more important for pile foundations than for spread footings. The reason for this is that there are no theoretical tools available which allow the engineer to solve for the value of skin friction on the pile while he is able to approximate by theory the ultimate load for a spread footing. Load tests are essential on a large project since it is the only method by which the foundation's safety factor can be determined.

The skin friction can be determined either of two ways by load tests. The first method is to test to failure by the standard vertical load test. If the failure is gradual, the failure load should be considered that which causes a 2-inch settlement. The test is made by jacking the load on the pile; the jack must be calibrated and react against either a loaded box or a steel beam anchored to other piles. In this way, the ultimate load (Q) can be determined. From chapter V, it is known that this ultimate load is equal to the sum of the point bearing load (Q_p) and friction load (Q_f). Ultimate skin friction per square foot is determined as follows:

$$Q = Q_p + Q_f$$

$$Q = Q_p + 2\pi r D_f f_s$$

or solving for f_s:

$$f_s = \frac{Q - Q_p}{2\pi r D_f} \tag{7.1}$$

Q_p can be approximated by equation (5.4) and the value of f_s determined.

The second method is by a tension or "pulling" test to failure. In this test, there is no end bearing and, thus, Q_f is equal to the ultimate load. Ultimate skin friction per square foot is determined as follows:

$$Q_f = 2\pi r D_f f_s$$

$$f_s = \frac{Q_f}{2\pi r D_f} \tag{7.2}$$

This equation is, of course, identical with equation (7.1) since $Q - Q_p = Q_f$.

Figures 7-1 through 7-17 present the results of all pile tests made by NACA; also included are photographs and sketches of pile test setups. On the pile-test graphs, the following information is furnished:

(1) Settlement load curve

(2) Boring log with Gow penetration value and pile length and cut-off indicated

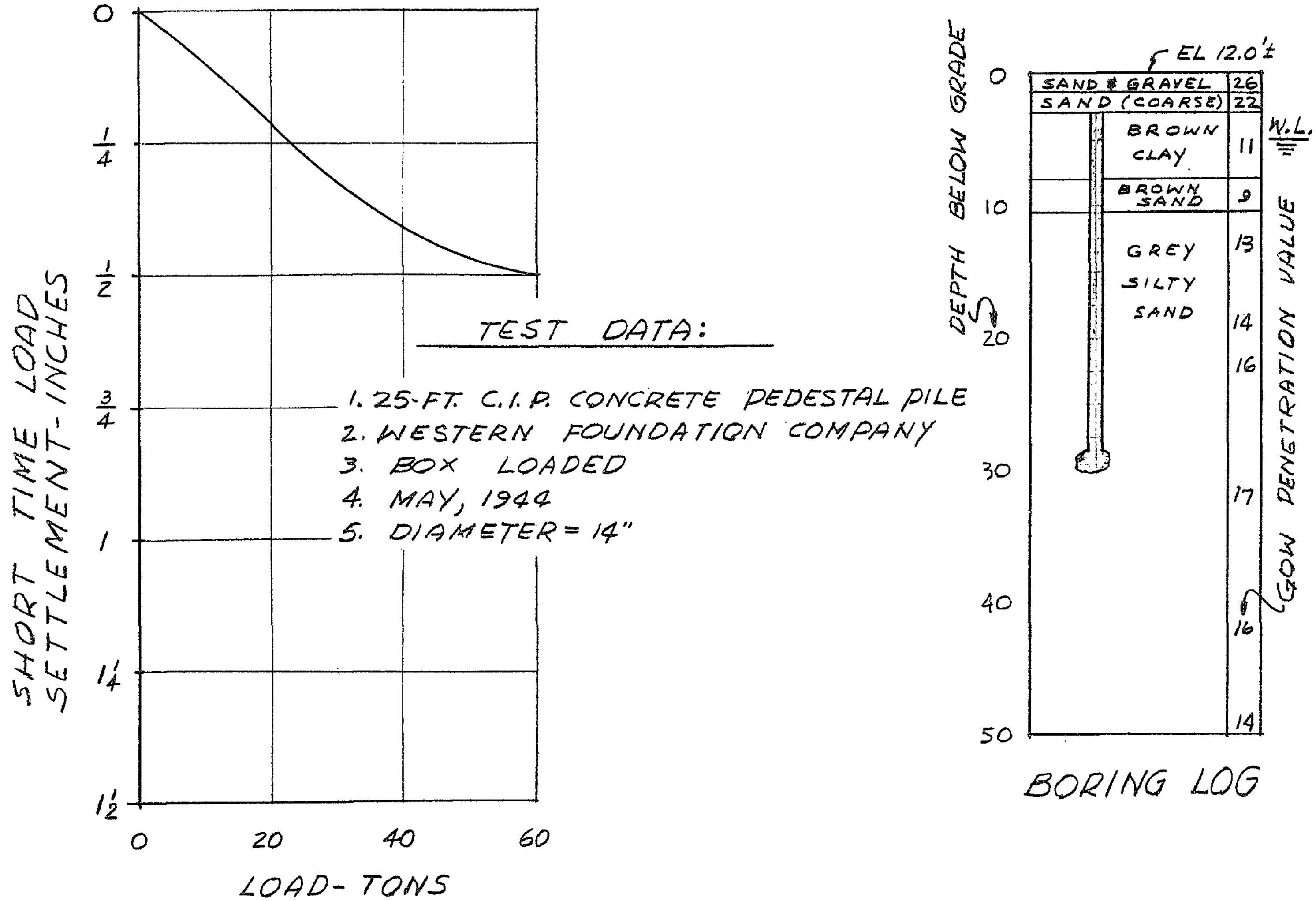


FIGURE 7-1

TEST PILE NO. 1--AIRCRAFT LOADS BUILDING

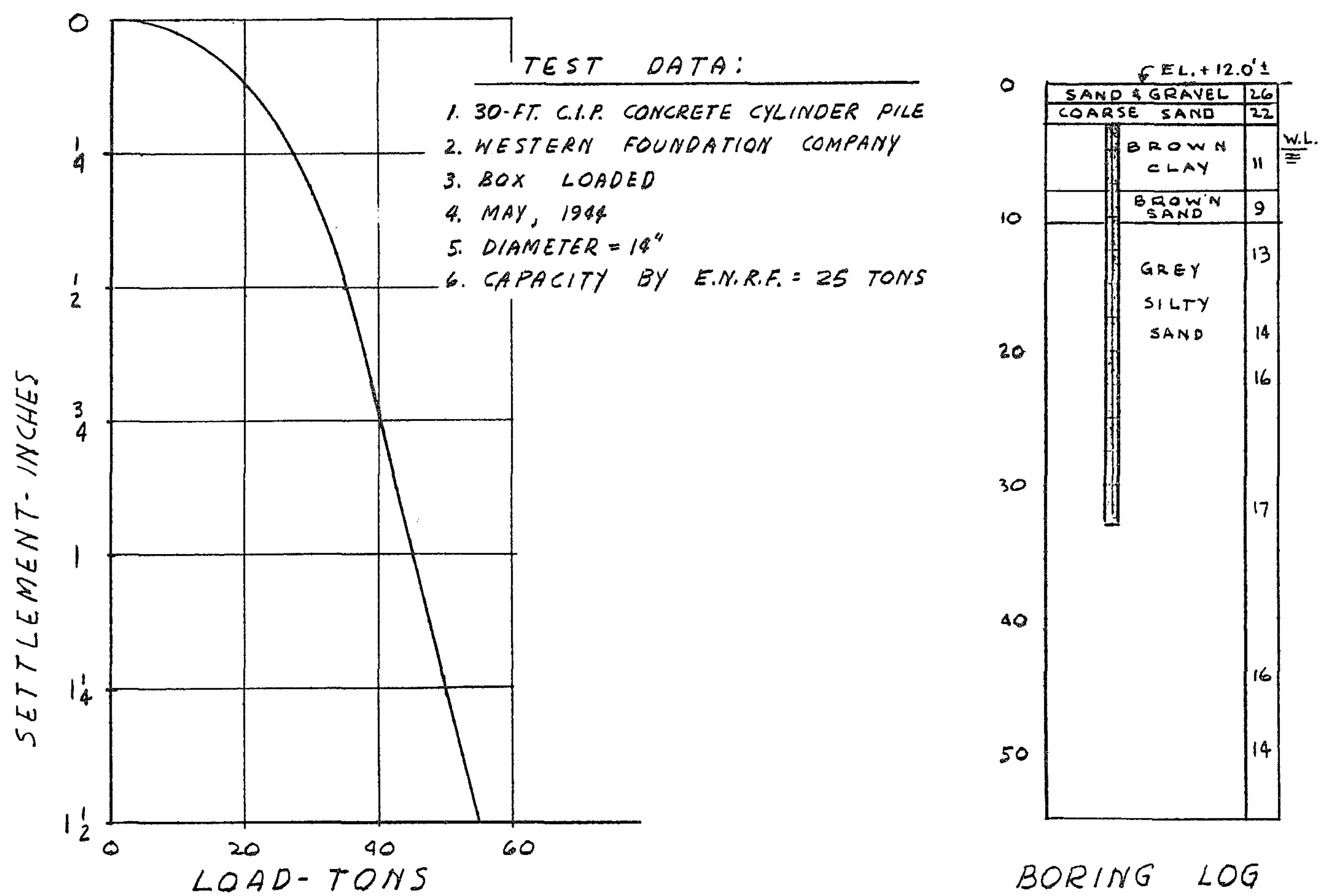
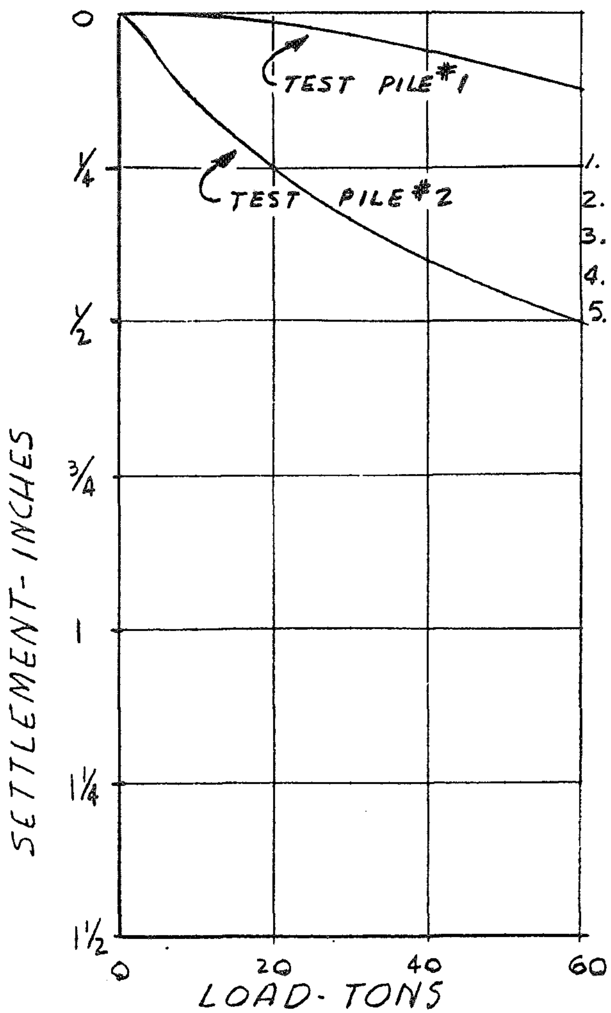


FIGURE 7-2

TEST PILE NO. 2--AIRCRAFT LOADS BUILDING



TEST DATA:

1. 40-FT. C.I.P. CONC. CYL. PILE
2. MAC ARTHUR PILE CORP.
3. BOK LOADED
4. MAY, 1944
5. CAPACITY BY E.N.R.F.
   TEST PILE #1 - 25 TONS
   TEST PILE #2 - 16 TONS

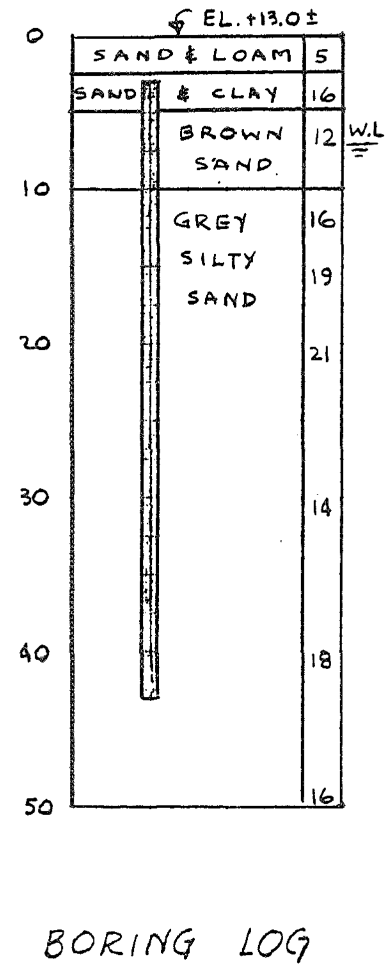
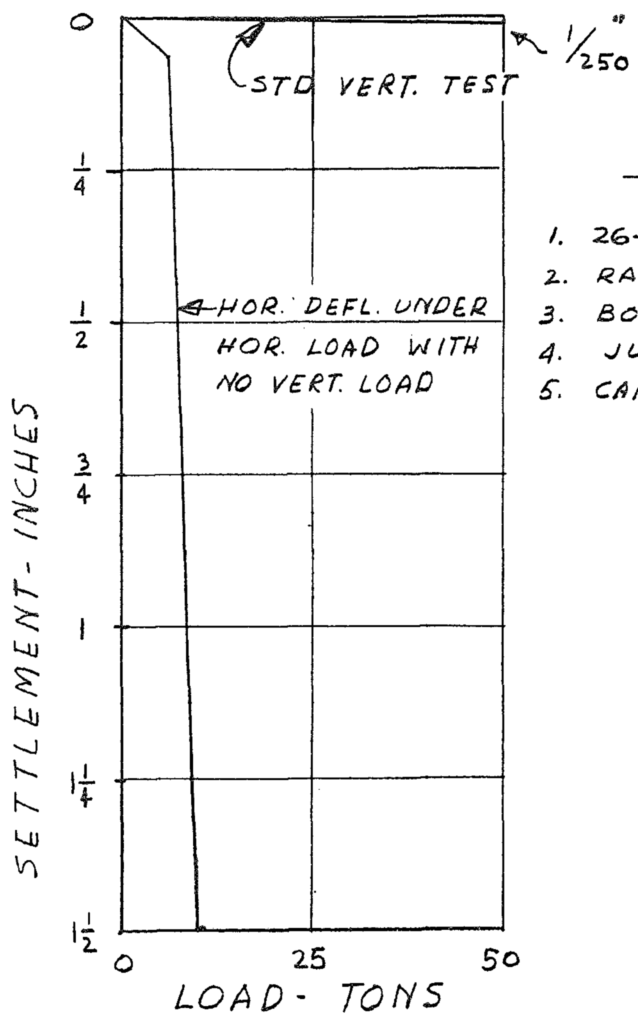


FIGURE 7-3

TEST PILES--INSTRUMENT RESEARCH LABORATORY



TEST DATA:

1. 26-FT. STD. RAYMOND PILE
2. RAYMOND PILE COMPANY
3. BOX LOADED
4. JULY, 1949
5. CAPACITY BY E.N.R.F. = 37 TONS

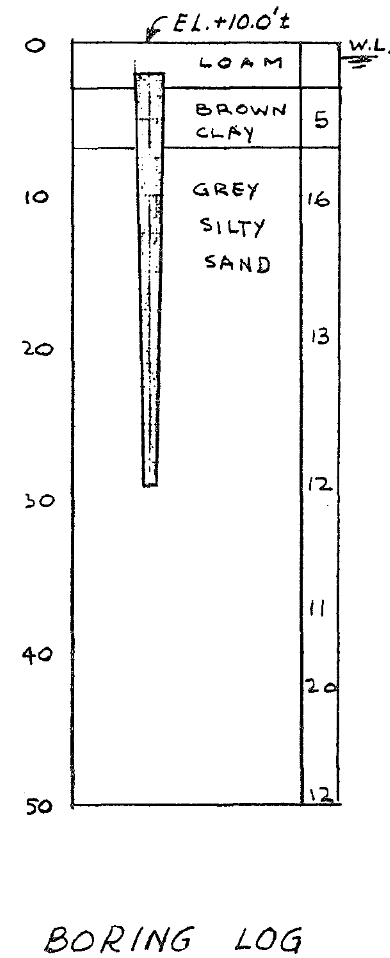
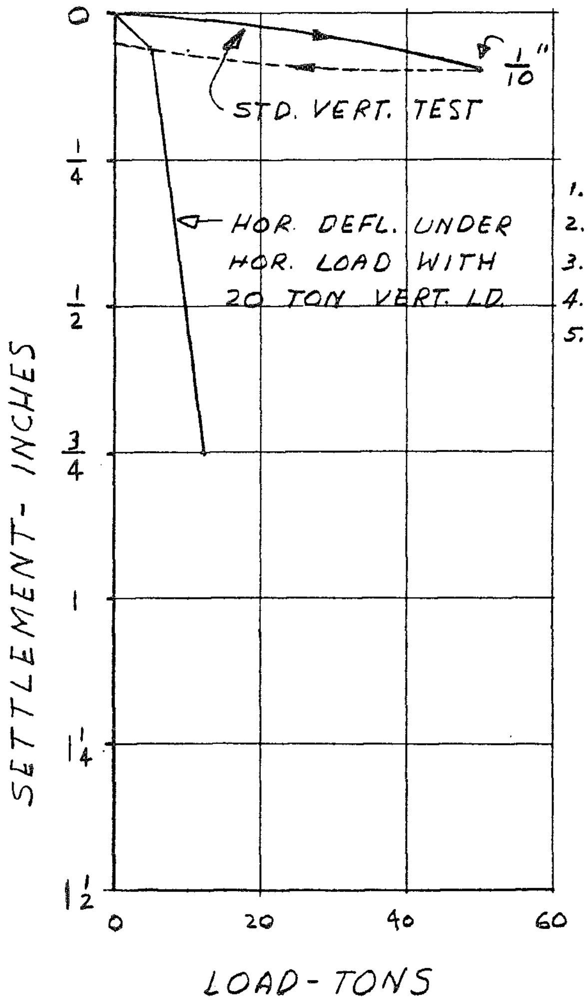


FIGURE 7-4

TEST PILE NO. 1--FLIGHT RESEARCH HANGAR



TEST DATA:

1. 22-FT. STD. RAYMOND PILE
2. RAYMOND PILE COMPANY
3. BOX LOADED
4. JULY, 1949
5. CAPACITY BY E.N.R.F. = 81 TONS

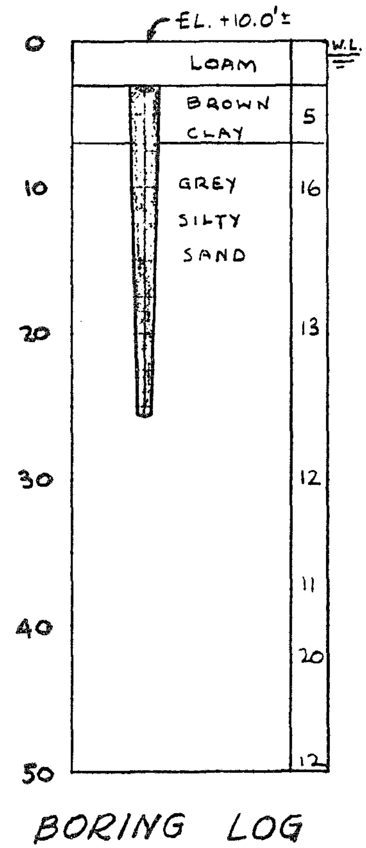


FIGURE 7-5

TEST PILE NO. 2--FLIGHT RESEARCH HANGAR

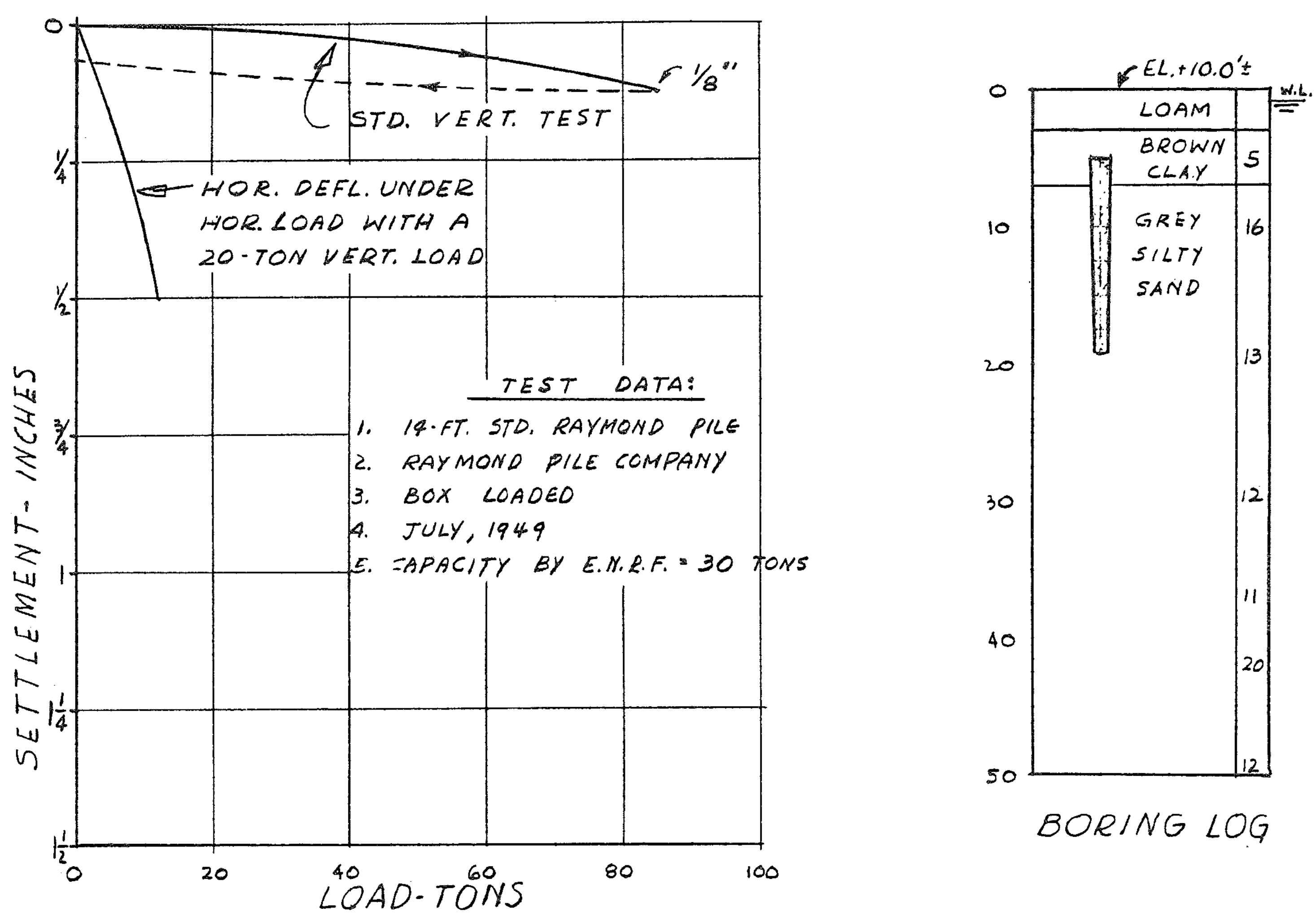


FIGURE 7-6

TEST PILE NO. 3--FLIGHT RESEARCH HANGAR

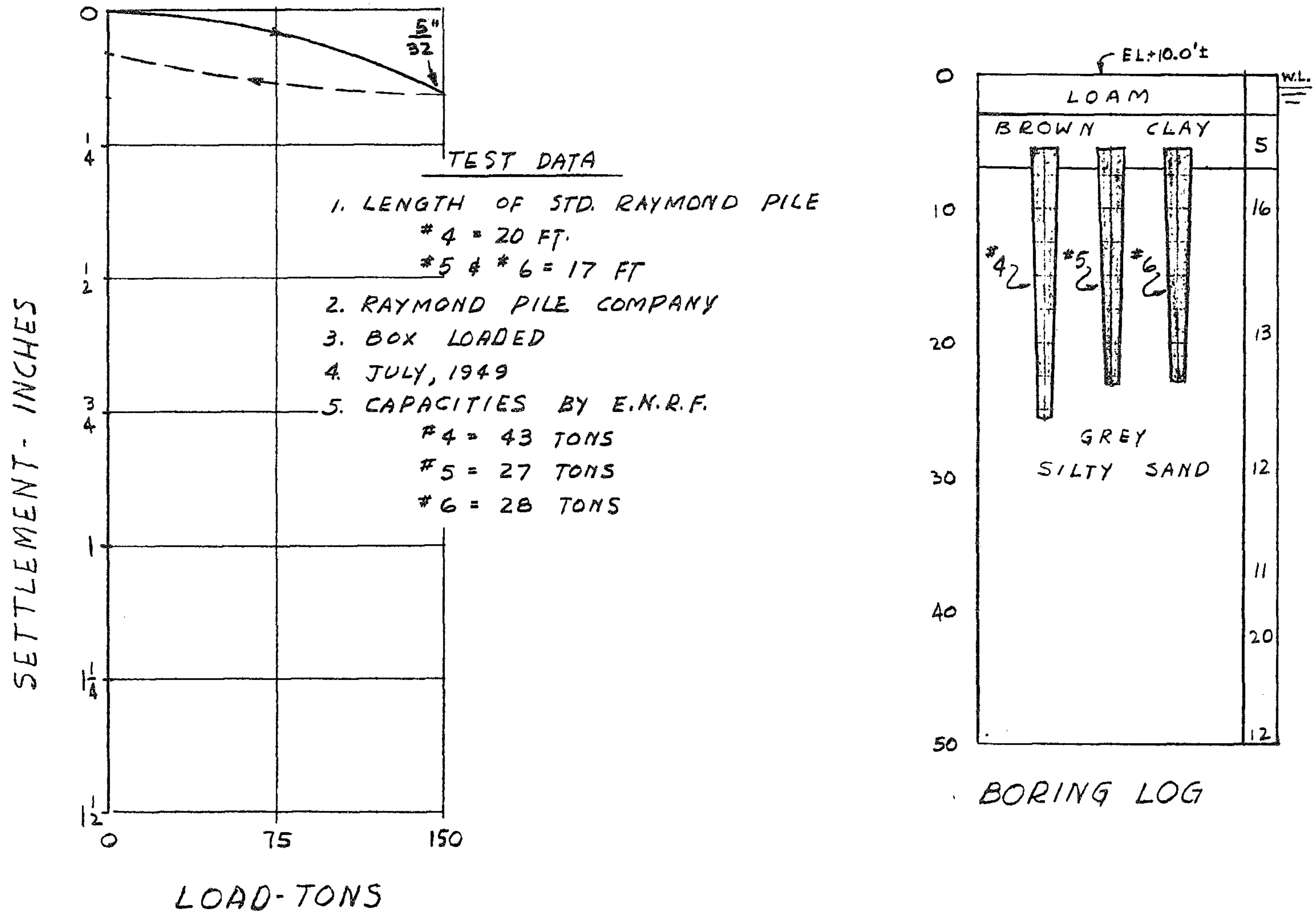


FIGURE 7-7

CLUSTER TEST--FLIGHT RESEARCH HANGAR

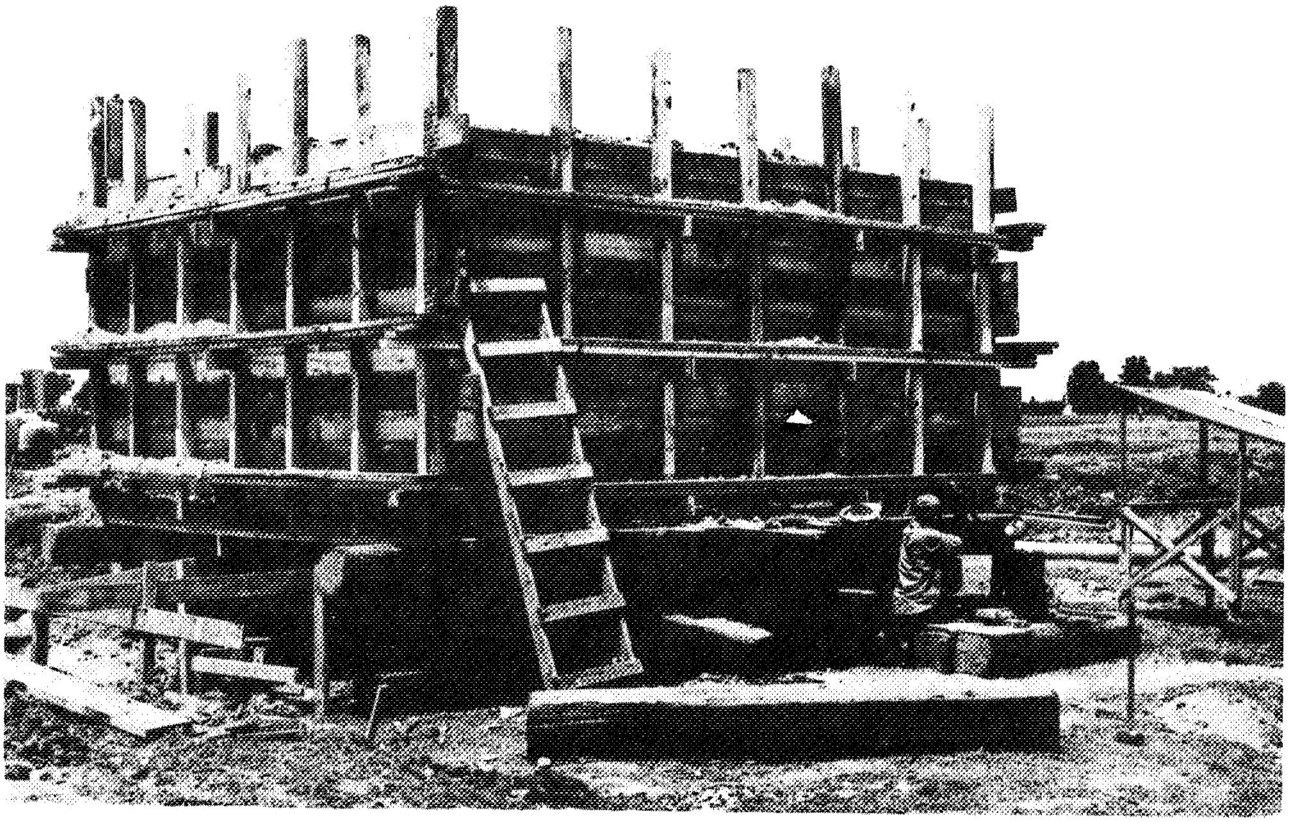


FIGURE 7-8

PILE TEST SETUP--FLIGHT RESEARCH HANGAR

NACA
L-61917

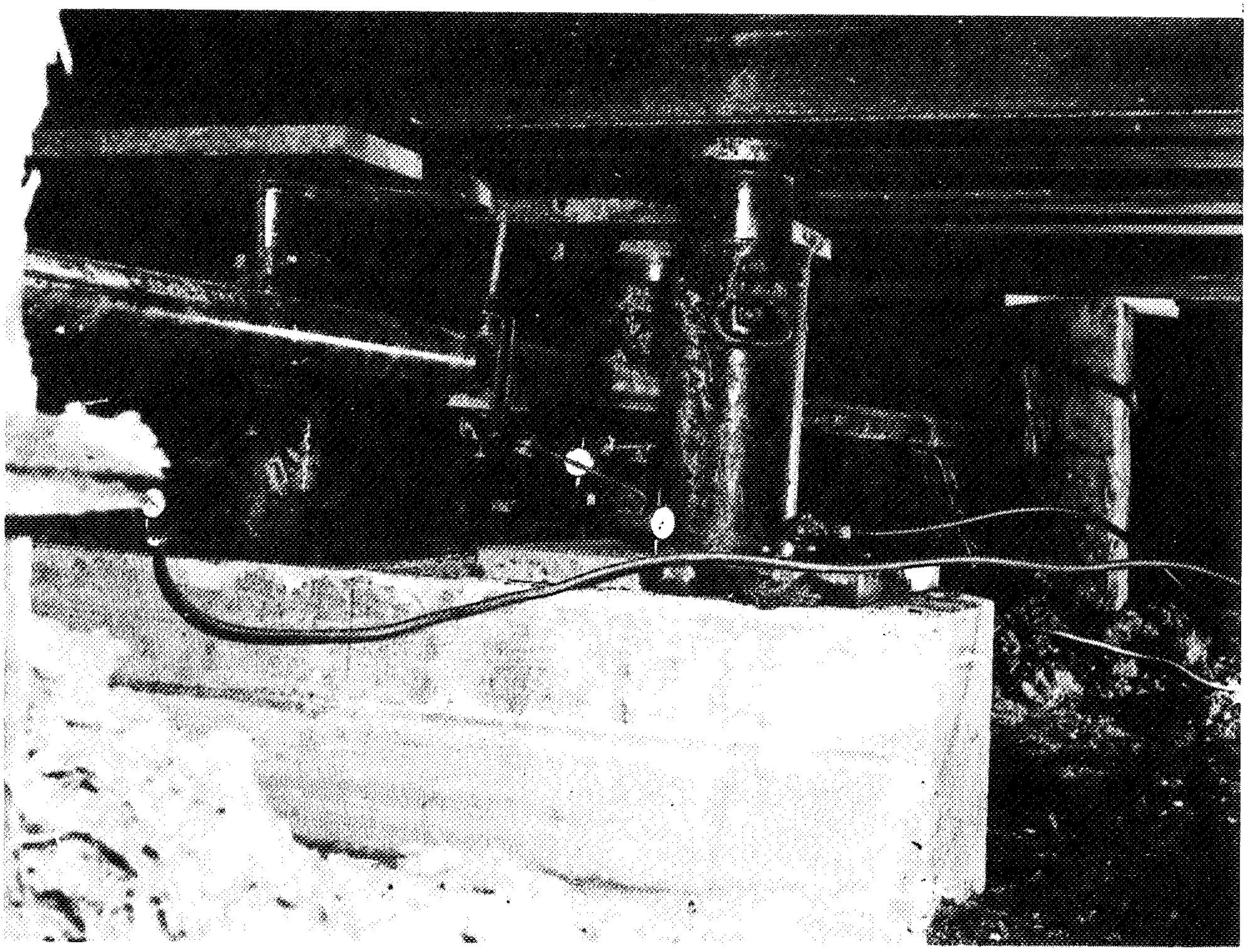


FIGURE 7-9

NACA
L-62106

VERTICAL TEST LOAD ARRANGEMENT--TEST PILES NOS. 4, 5, AND 6--
FLIGHT RESEARCH HANGAR

FIGURE 7-10

LATERAL LOAD TEST SETUP FOR TEST PILE NO. 1--
FLIGHT RESEARCH HANGAR

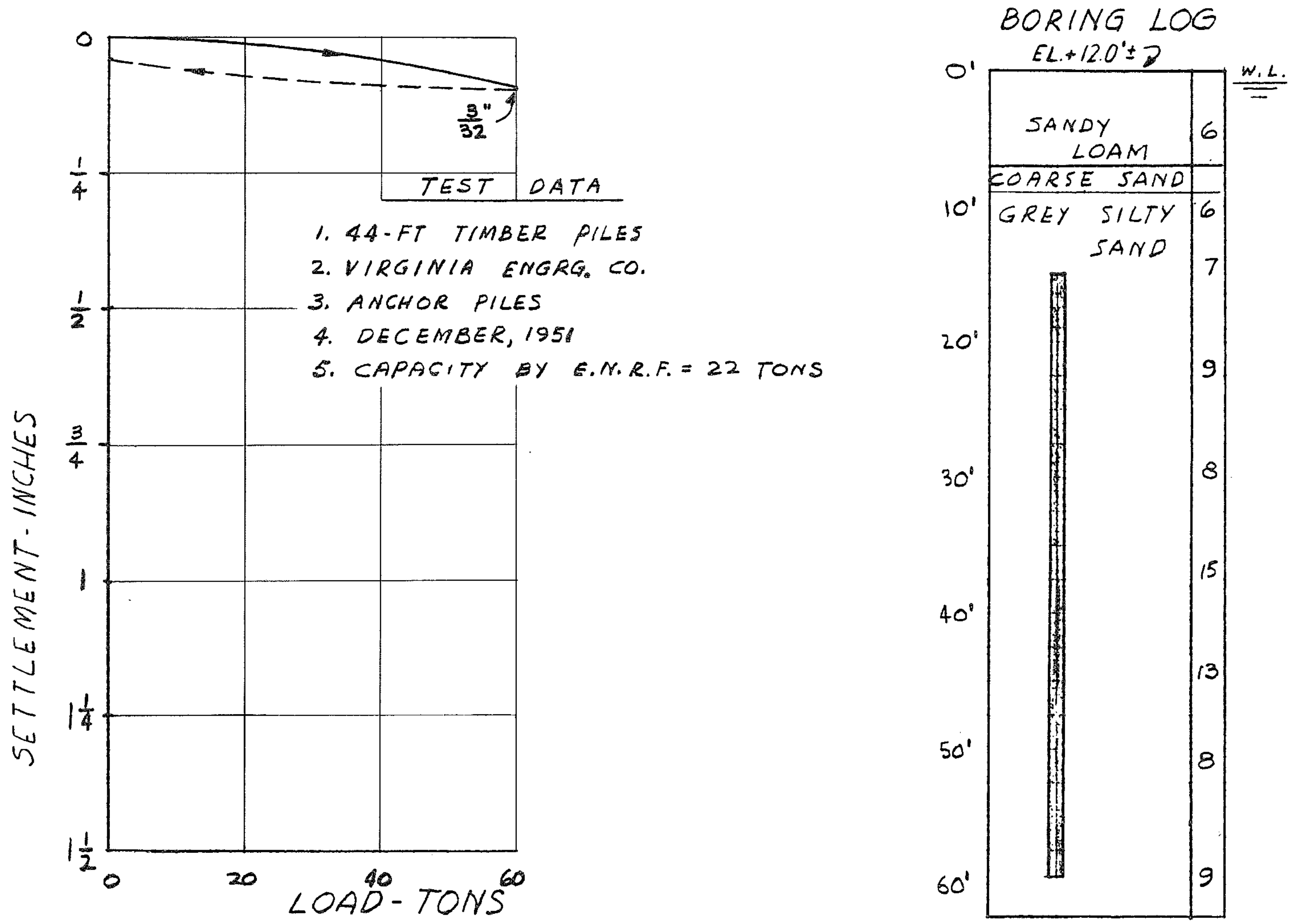


FIGURE 7-11

TEST PILE NO. 1, 4-FOOT BY 4-FOOT U.P.D.T.

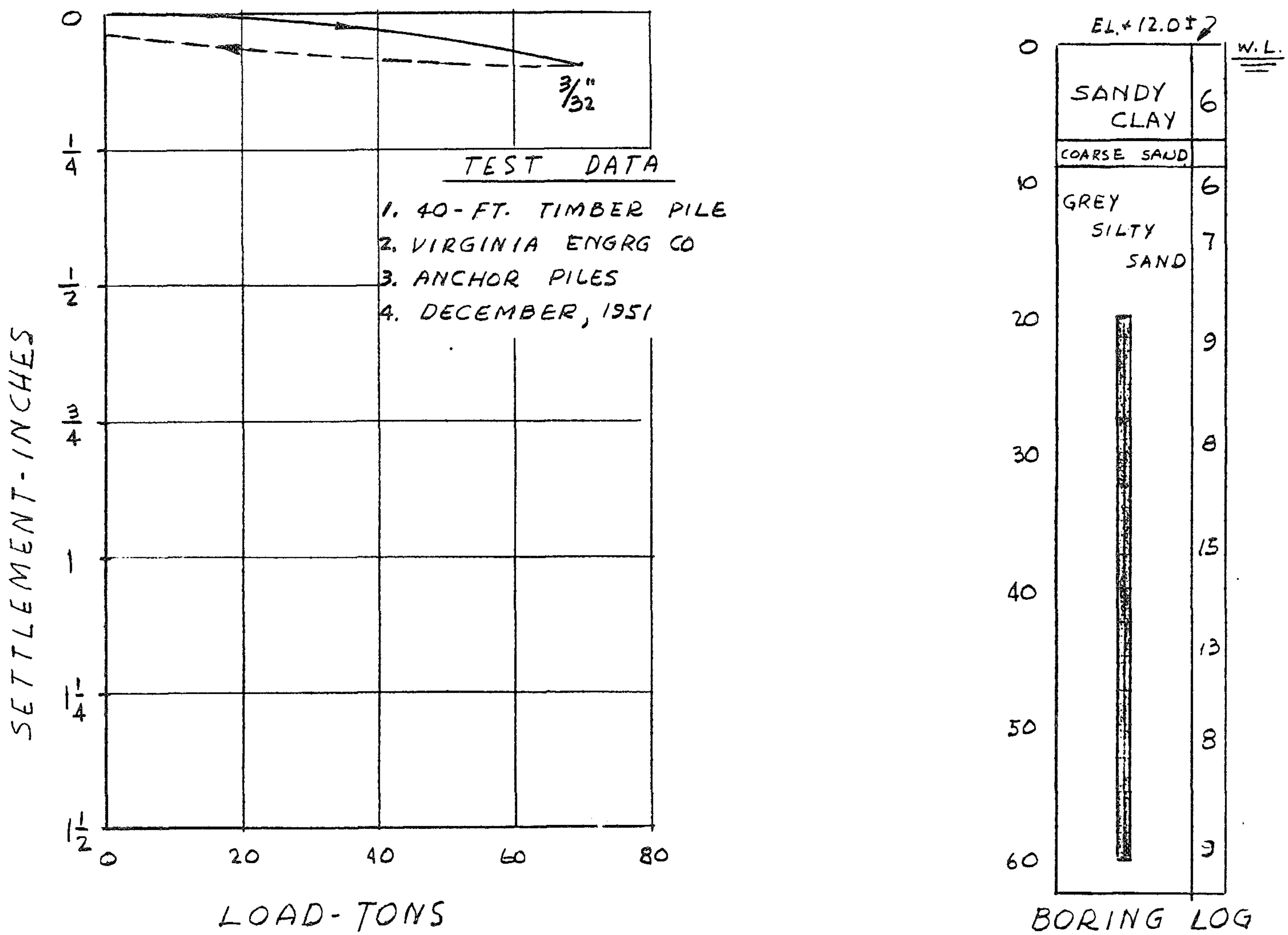


FIGURE 7-12

TEST PILE NO. 2, 4-FOOT BY 4-FOOT U.P.D.T.

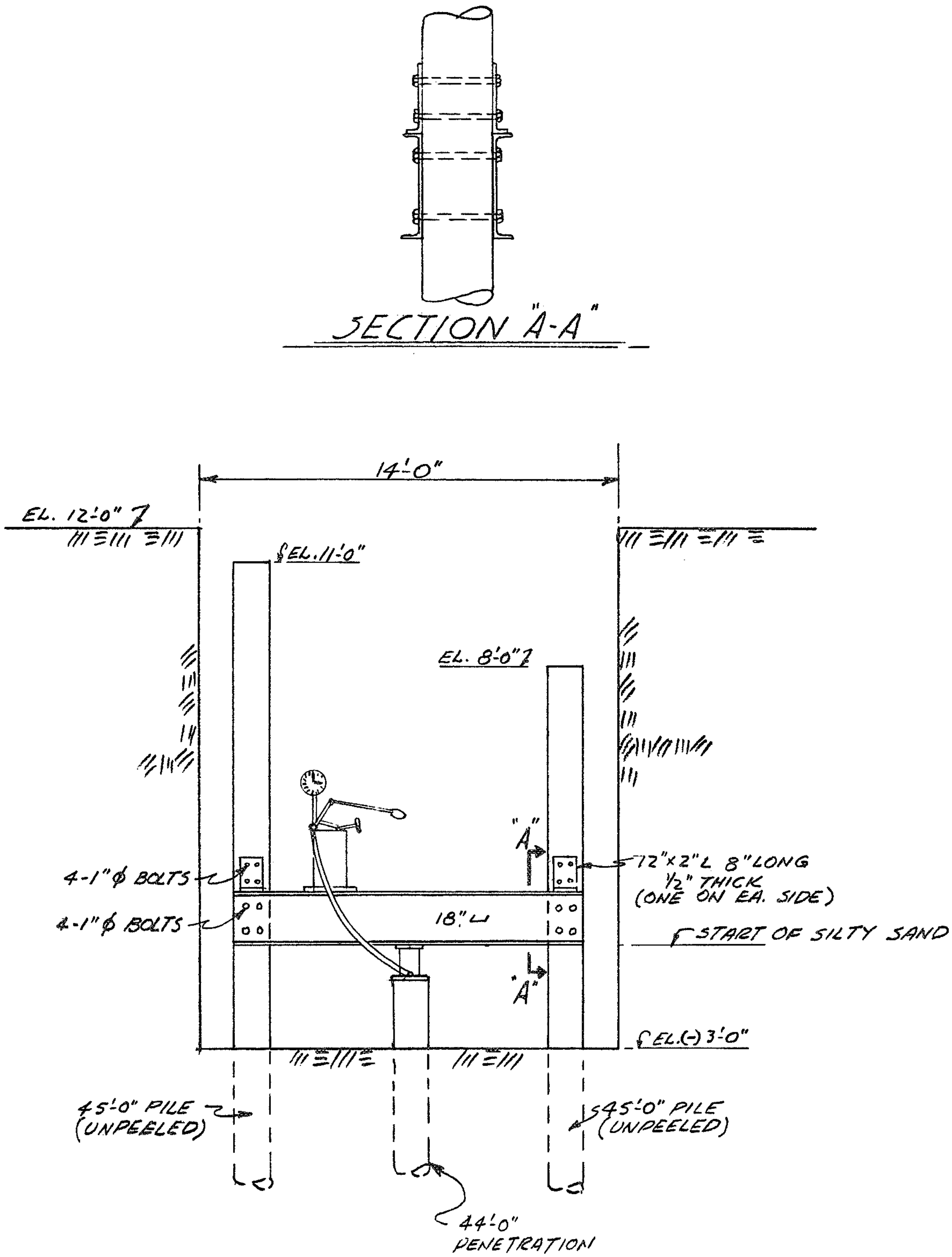
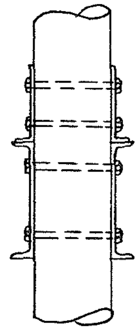


FIGURE 7-13

GENERAL SETUP, TEST PILE NO. 1, U.P.D.T.



SECTION "A-A"

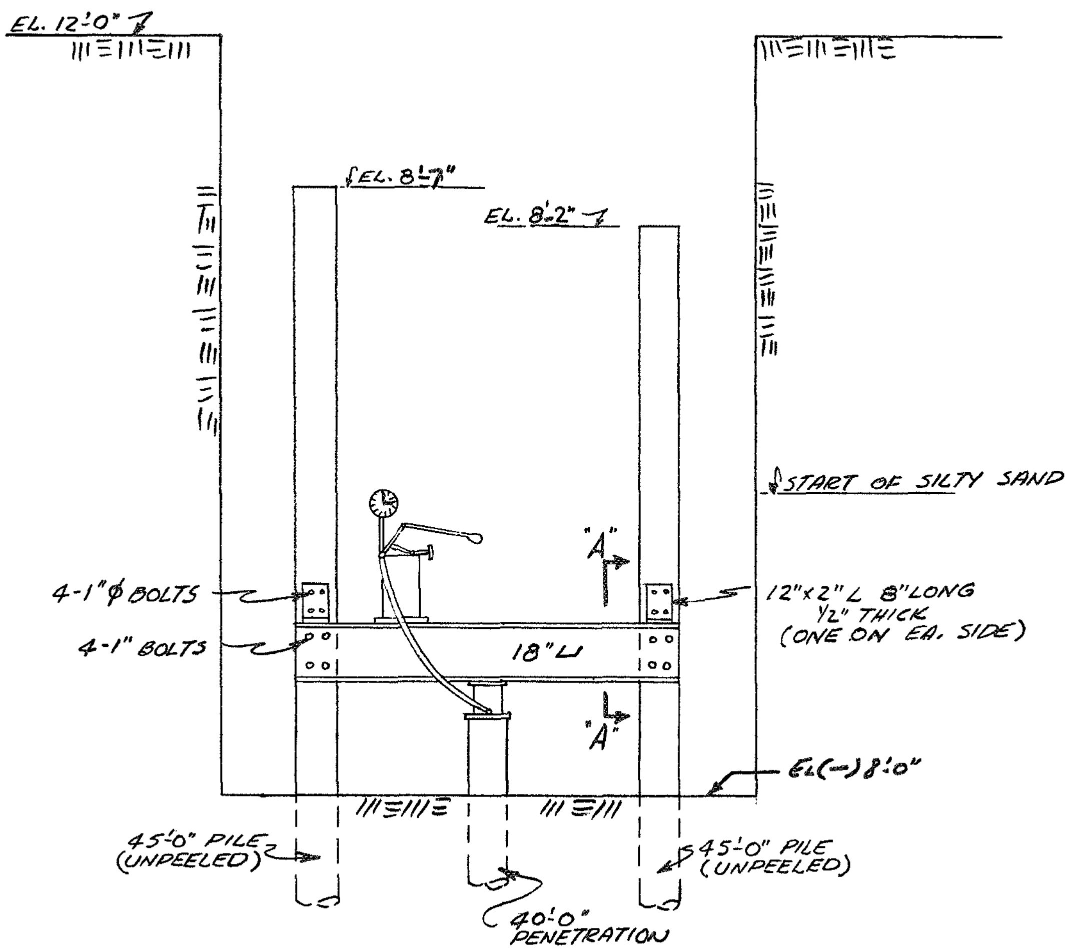


FIGURE 7-14

GENERAL SETUP, TEST PILE NO. 2, U.P.D.T.

FIGURE 7-15

TEST PILE SETUP, U.P.D.T.

NACA
L-73472

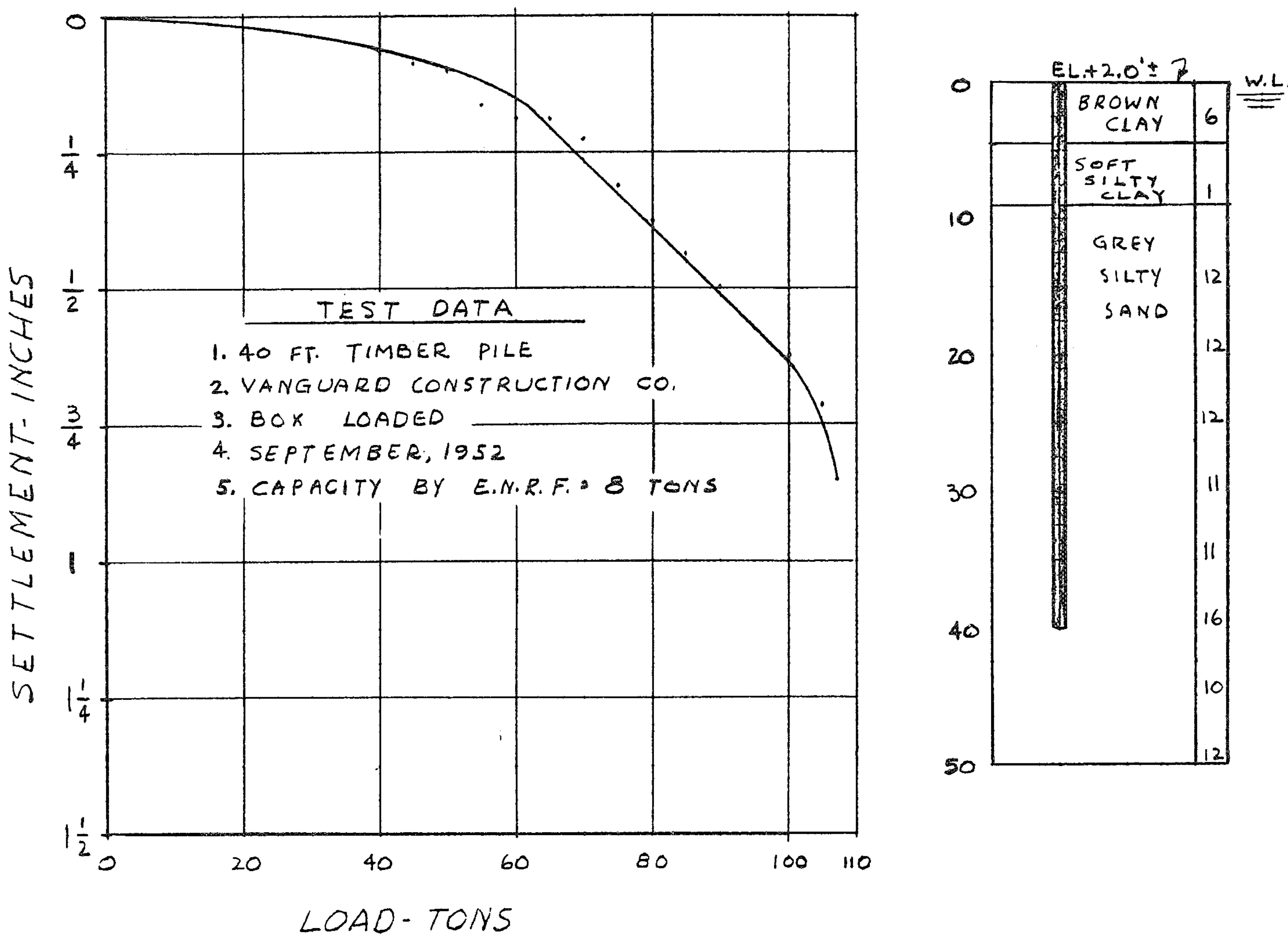


FIGURE 7-16

TEST PILE--WATER TOWER FOUNDATION

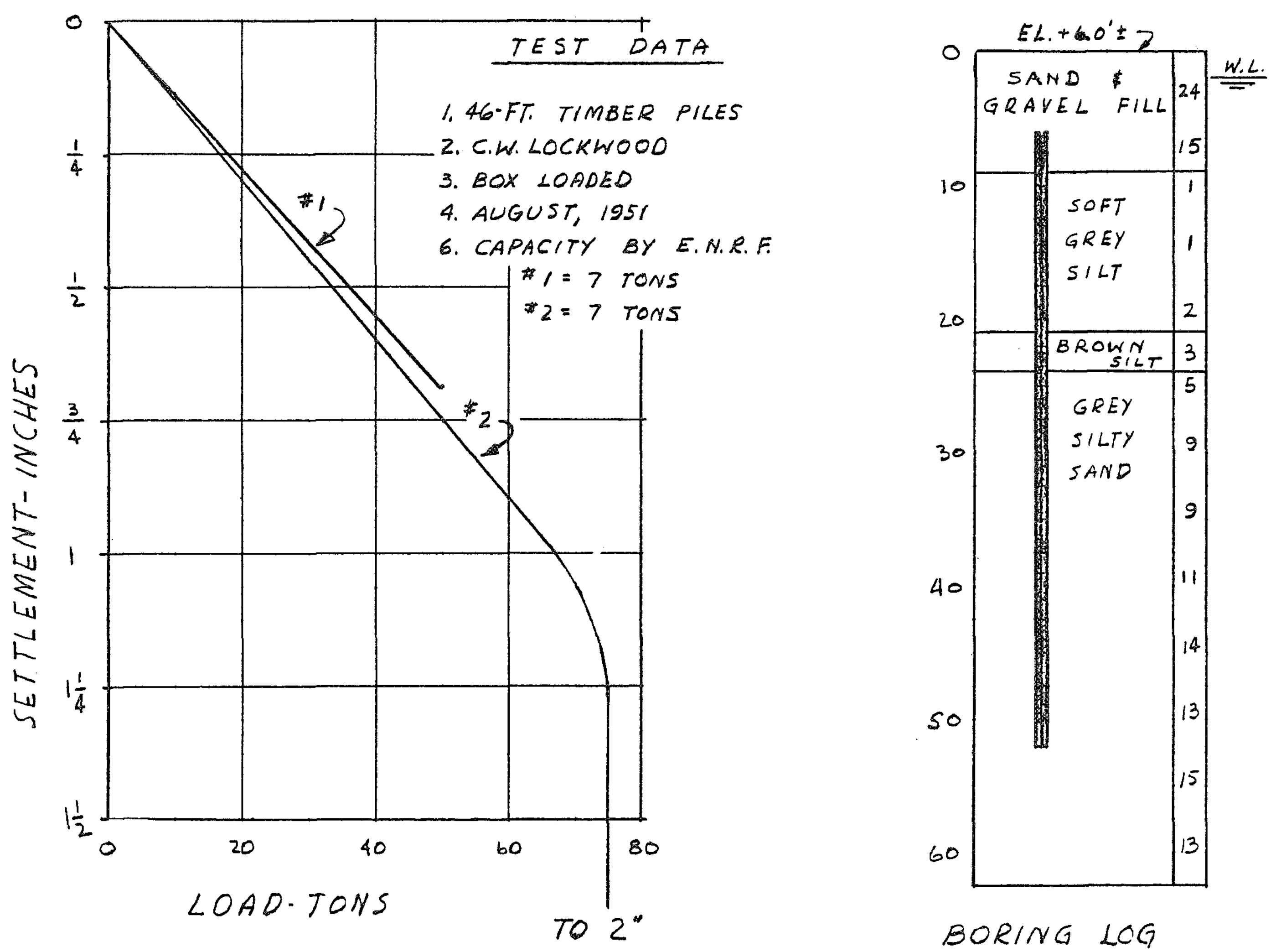


FIGURE 7-17

TEST PILES--8-FOOT OFFICE BUILDING ADDITION

(3) Type of pile

(4) Contractor installing piles

(5) Method of loading

(6) Date tested

(7) Capacity by the Engineering News Record Formula when driving information is available

To compute f_s, it is necessary that the pile test be carried to failure; it is regretted that all piles were not so tested. However, either because of testing to failure or because of a high capacity test, much information can be obtained from the following pile tests:

(1) Test Pile No. 2 - Aircraft Loads Building (Fig. 7-2)

(2) Test Pile No. 3 - Flight Research Hangar (Fig. 7-6)

(3) Test Pile - Water Tower Foundation (Fig. 7-16)

(4) Test Pile No. 2 - 8-Foot Office Building Addition (Fig. 7-17)

The following results are obtained for the above:

Pile	Q	Q_p	Q_f	$2\pi r D_f$	f_s
TP No. 1, ALB	60 tons	14 tons	46 tons	95 s.f.	0.48 t.s.f.
TP No. 3, FRH	85	12	73	44	1.66
TP, Water Tower	107	14	93	126	.73
TP No. 2 8-Ft OB	75	18	57	144	.40

Thus, a maximum value of f_s = 3320 p.s.f., a minimum value of f_s = 800 p.s.f. and an average value of 1640 p.s.f. has been determined.

A plot of ultimate pile capacities for various lengths based on the average test-determined value of skin friction is presented as Figure 7-18.

An inspection of Figure 7-18 reveals that the piles are primarily the "friction" type. For a 40-foot pile, the division is approximately 87 per cent friction and 13 per cent end bearing.

Two plate tests were made in 1947 at the 16-foot tunnel site to determine an allowable soil pressure for the footings of an auxiliary electrical building. The results of these tests are presented in Figure 7-19.

Field plate tests do not give results as reliable as those obtained from a pile test. This is primarily because a plate does not approach a footing in size and a full-scale test is seldom run because of expense involved. Besides the size factor itself, it is possible that the tests, being made on such small areas, are not run on a true representative soil condition.

There is, however, an important piece of information to be noted from the tests. Both the mode of failure and the characteristics of the load-settlement curve reveal that the soil fails by "general" shear rather than "local" shear.

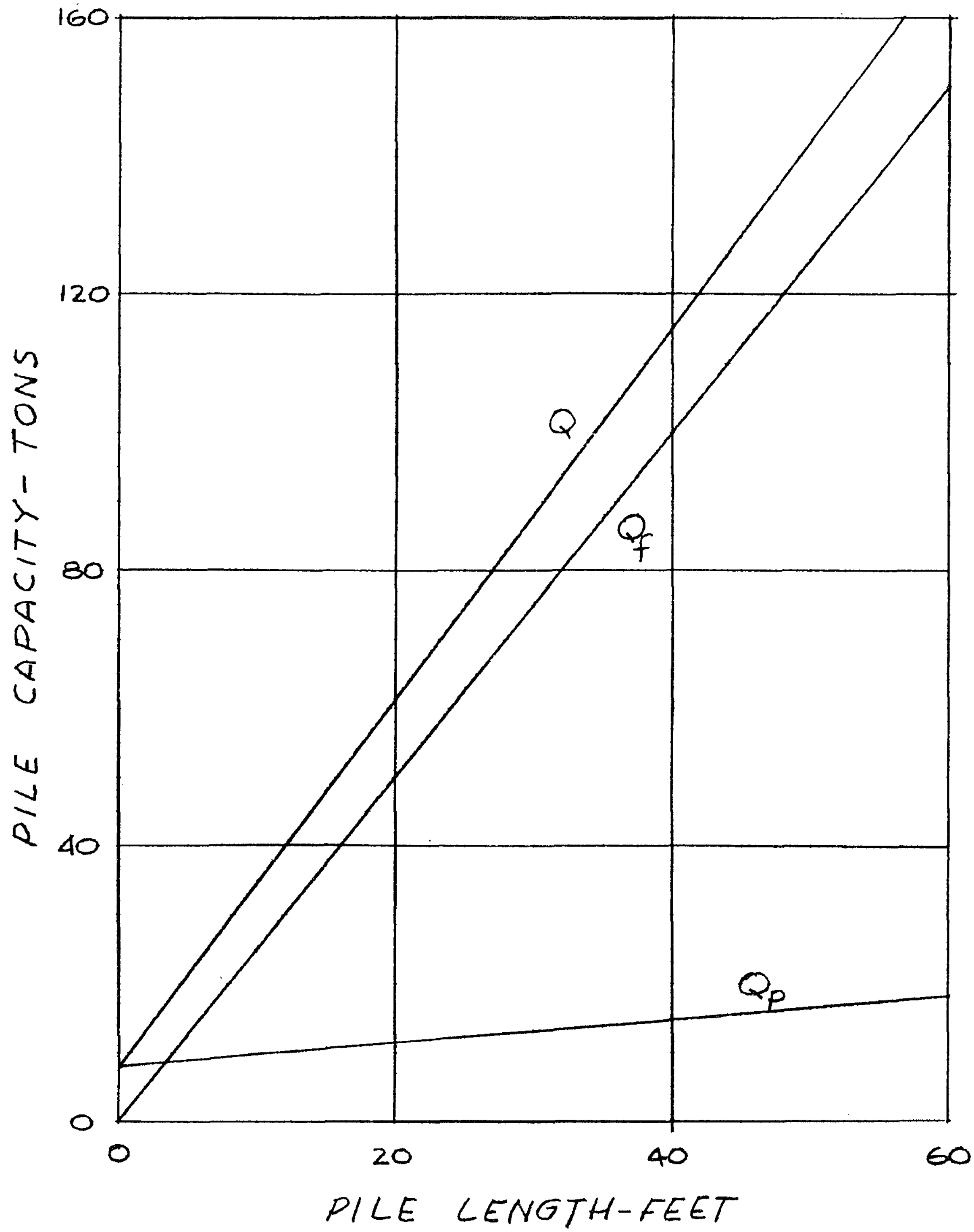


FIGURE 7-18

ULTIMATE PILE LOADS BASED ON TEST-DETERMINED SKIN FRICTION VALUE

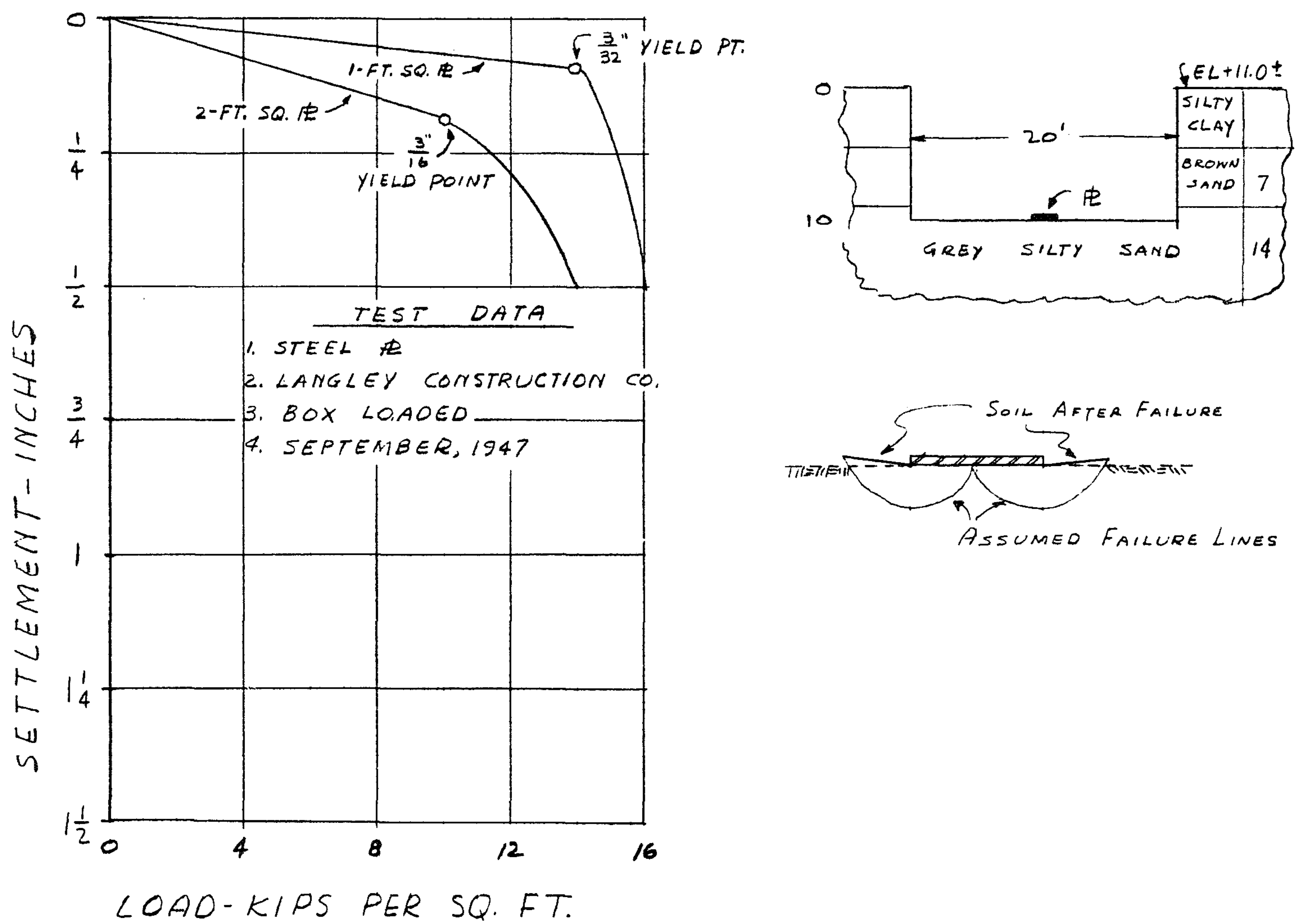


FIGURE 7-19

PLATE TESTS--16-FOOT HIGH-SPEED TUNNEL

This is important since to make a theoretical analysis it is necessary to assume a mode of failure. The analysis made in chapter V was based on a "general" shear failure.

Using the results of the load tests, average f_s for piles and minimum yield point for spread footings, and applying a safety factor of 2-1/2, the following allowable bearing values are obtained:

(1) For spread footings

q_a = 2 tons per square foot

(2) For a 40-foot pile

Q_a = 45 tons

CHAPTER VIII

ACTUAL SETTLEMENT BY FIELD RECORD

Periodic settlement readings are taken of NACA facilities for two reasons. The primary reason is to anticipate trouble so that corrective steps can be taken prior to damage. Secondarily, these readings will enable the designer to reduce his safety factor for unknowns and thereby reduce the cost of foundations.

Settlement can be divided into three components; they are:

(1) Elastic settlement which is proportional to the load by Hooke's law

(2) Primary consolidation, which is the reduction of voids due to the squeezing out of the water from the increased load

(3) Secondary consolidation which is the gradual adjustment of the soil grains due to the reduction in voids

Extreme caution was exercised in the placing of the bench marks on the foundation and, wherever possible, elevations were established prior to all loading. In both cases to be presented, the settlement readings represent the total of the three components.

Approximate formulas and theory are available to evaluate the first two components but no progress has been made theoretically with regard to secondary consolidation.

Secondary consolidation's characteristics can be investigated only by observation and settlement readings over a period of time. One important fact has been uncovered; it is that secondary consolidation settlement varies with the log of time. This fact is of great value since it becomes evident that, after settlement readings are kept over a period of time, future settlement can be estimated.

Figure 8-1 is the settlement field record of the pile foundation investigated theoretically in chapter VI. It indicates that settlement is progressing linearly with the log of time, that is, secondary consolidation. Settlement to date is approximately 1/2 inch; extrapolation of the graph to estimate future settlement is given on Figure 8-2. Settlement to date is roughly one-half the expected 25-year settlement. It should be noted that the elastic and primary consolidation settlements are negligible.

The settlement field record of the Freon compressor foundation, also investigated in chapter VI, is presented as Figure 8-3. The curve shows similar characteristics to the previously presented pile foundation's graph. The one exception is the variation from the straight line settlement

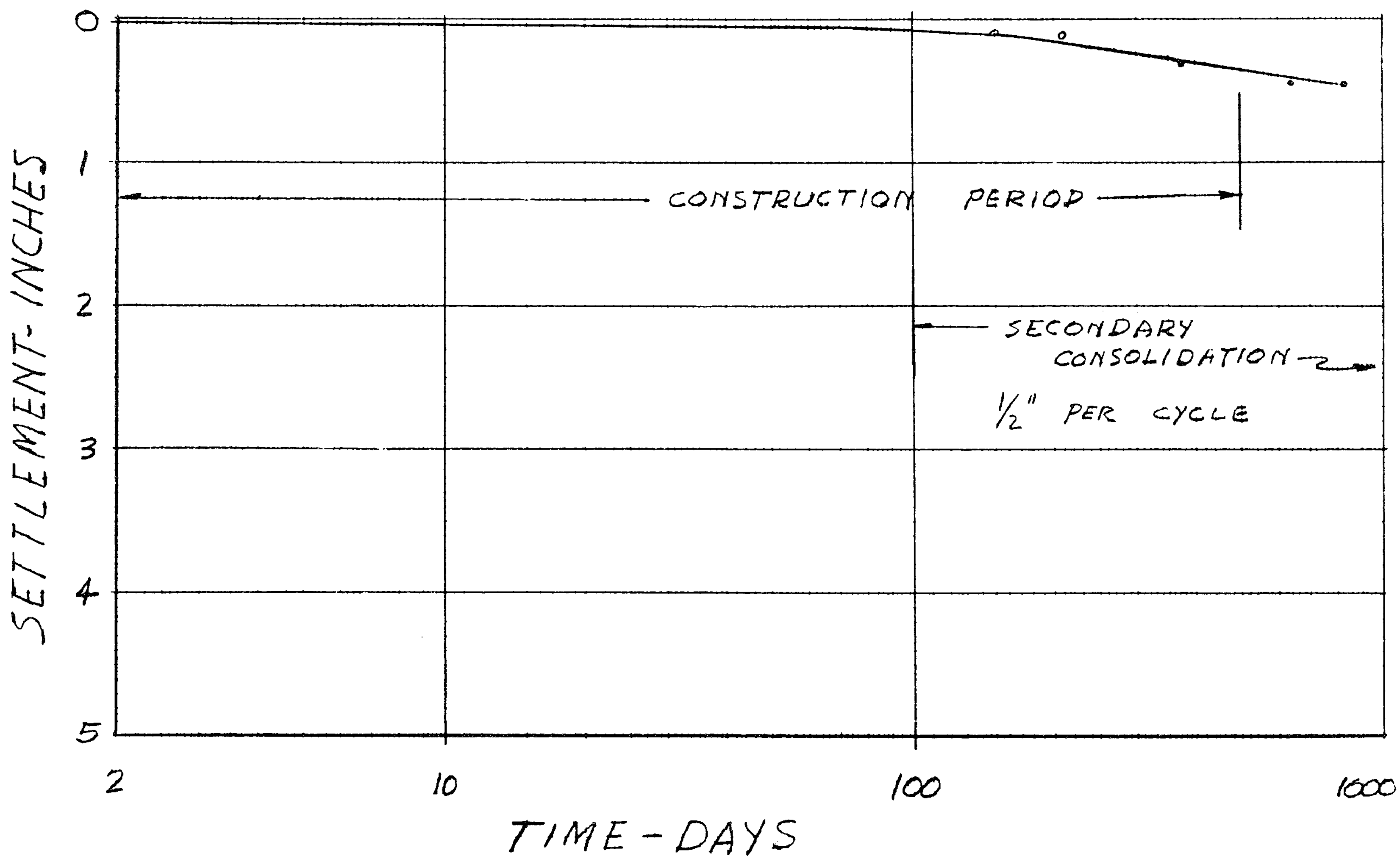


FIGURE 8-1

SETTLEMENT OF PILE FOUNDATION--FIELD RECORD

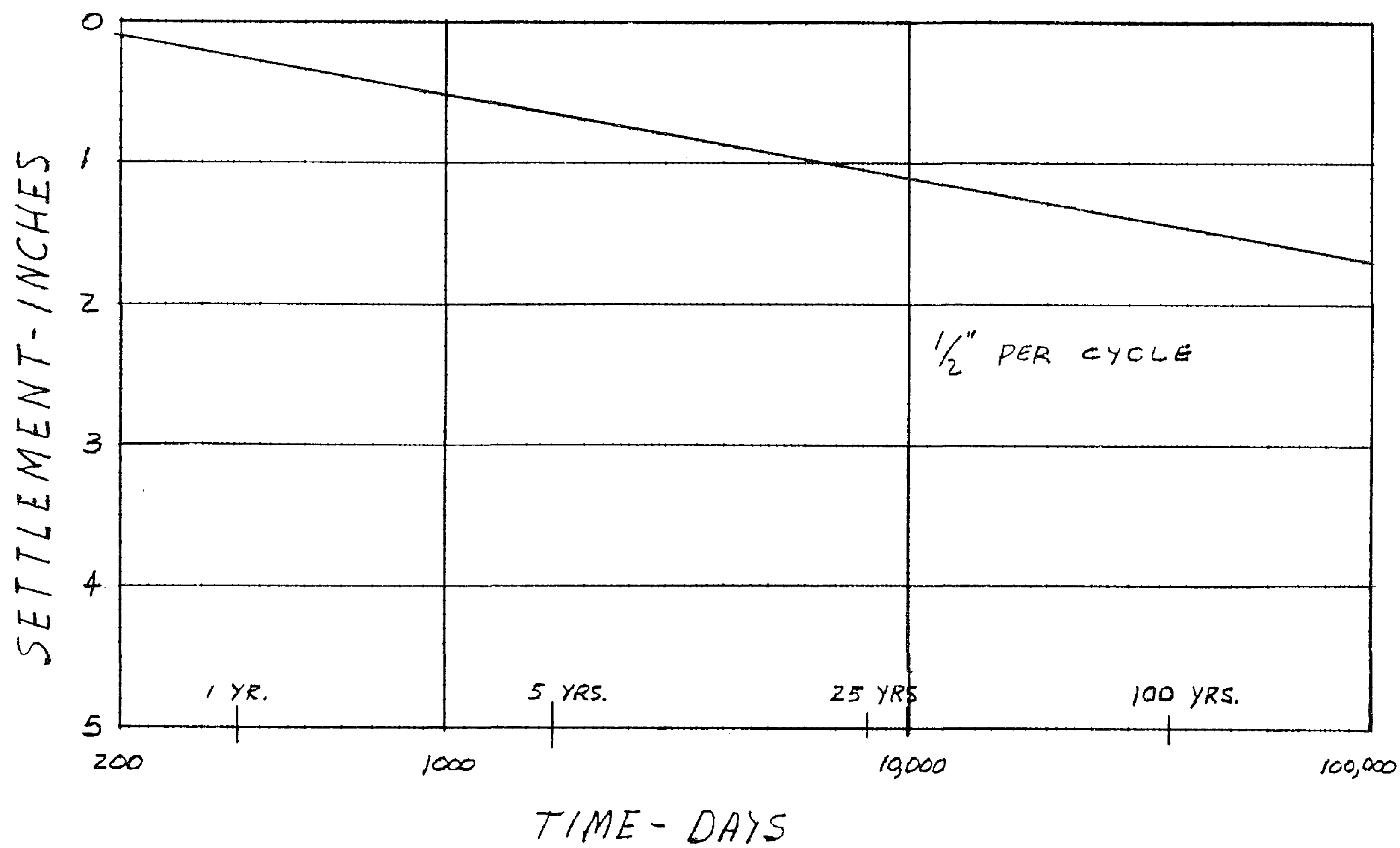


FIGURE 8-2

FUTURE SETTLEMENT OF PILE FOUNDATION

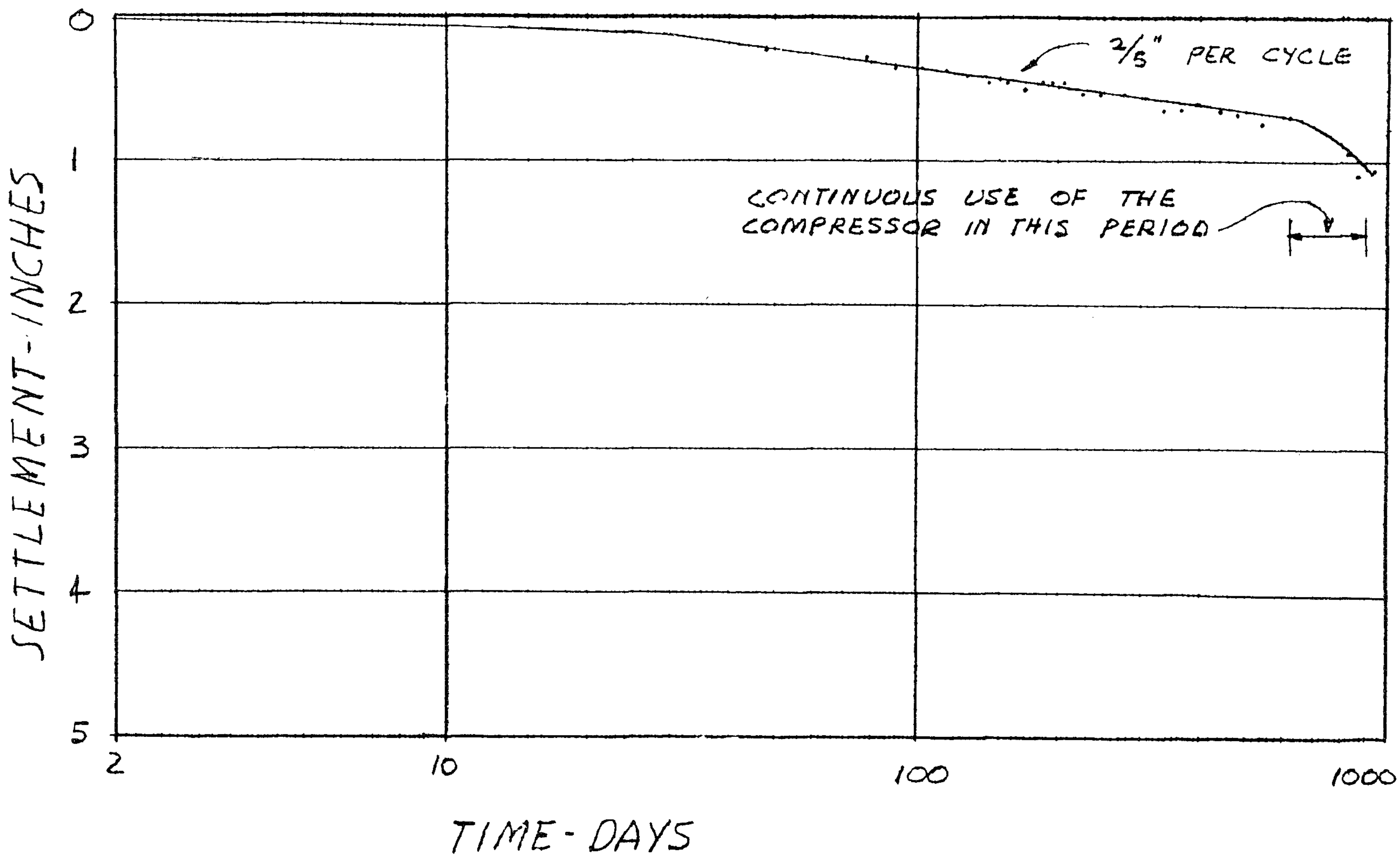


FIGURE 8-3

SETTLEMENT OF COMPRESSOR FOUNDATION--FIELD RECORD versus log time relationship at approximately 700 days. This may be explained by vibratory effects since the compressor was in constant use during that period. A tendency to return to the prior slope has been noted recently while the compressor was inoperative but it is too soon to draw any conclusions.

To sum up, field settlement readings to date have indicated that primary consolidation is unimportant while secondary consolidation has to be carefully watched and provided for in the design.

CHAPTER IX

CORRELATION OF THEORY AND TEST RESULTS--BEARING CAPACITY

It might be provident at this point to review briefly the salient points which have been established previously in this thesis with regard to load-carrying characteristics. It has been emphasized that only certain problems have been solved theoretically and that these require judicious use of laboratory results in order to obtain even an approximately accurate answer. Other problems, beyond the realm of present theory, can be handled only by an "educated" guess; laboratory tests, however, may serve as a basis for the guess. Field tests are regarded as the acid test from which there is no repeal; thus, no theory is rigorously correct but serves as an estimating basis only.

The design of spread footings are in the first category, that for which there is theory available which should give a reasonably accurate forecast of its bearing capacity. Friction pile foundations are in the second category for which no theoretical solution has been developed and, thus, a design basis can be obtained only by "educated" guessing.

There are currently three methods in general use for estimating the carrying capacity of a friction pile.

The most used and misused are various pile driving formulas. This method is a holdover from the pre-soil mechanics age and continues in favor by inertia and ease of use; an investigation of this method is contained in chapter X.

Despite the lack of the development of a theoretical analysis for friction piles, some important progress has been made. Recently, load tests have been made on piles to determine the distribution of skin friction over the depth of a pile. These tests, by means of strain gages placed at varying depths reveal that the skin friction is practically uniform (reference 3).

The value of the ultimate skin friction which the soil surrounding the pile can withstand is the only obstacle in way of a complete theoretical analysis; there is, however, no way to evaluate skin friction by laboratory test. Hence, the other two methods of estimating the pile's capacity both guess a value of ultimate skin friction. The most reliable method is to assume the value of skin friction equal to soil's cohesion value determined by the tri-axial shear test. This method was used in chapter X and is conservative since it makes no allowance for the increased bearing capacity due to the granular frictional properties of the soil. The second method is to pick a skin friction value for the soil's classification from tables contained in texts of soil mechanics. This method, while greatly preferable to the dynamic pile formula method, is not used extensively mainly because those who are familiar with it are also in a position to use the first method which is more reliable.

For evaluation of the accuracy obtainable from theory for use in spread footing design, the results of plate tests in chapter VII can be utilized. These results show that actual failure occurred at approximately 16 kips per square foot. The following results were, thus, obtained by test:

For a 1-foot plate

$$Q_{db} = 8 \text{ tons}$$

For a 2-foot plate

$$Q_{db} = 32 \text{ tons}$$

The theoretical analysis made in chapter V resulted in the following predictions:

For a 1-foot plate

$$Q_{db} = 11 \text{ tons}$$

For a 2-foot plate

$$Q_{db} = 44 \text{ tons}$$

It may be noted that the soil in actual field test failed prior to reaching its theoretical ultimate load.

This was to be expected because, as pointed out in chapter V, theory assumes that plate remains horizontal without tilting, whereas, in reality, all foundations tilt to some extent which reduces its bearing capacity. However, this is a recognized fact and is taken into account in choosing the safety factor.

Supposing, for purposes of illustration, we take the theoretical failure load of 11 tons per square foot and apply a safety factor of 2-1/2. This results in a design allowable load of 8800 pounds per square foot. Reference to Figure 7-19 shows this to be in the elastic range of the soil as revealed by the load test. It has, thus, been shown that theoretical analysis, while not resulting in an exact solution, does give a satisfactory approximate answer which is all that can be expected.

The estimates of pile capacity obtainable from quasi-theoretical methods will now be compared with the results obtained by load tests.

A table in reference 13 gives a range of ultimate skin friction values of 400 to 1000 pounds per square foot of contact area for piles embedded in sandy silt, the closest classification to silty sand given. Figure 9-1 graphically depicts the comparison of the results obtained by using these empirical values with the results obtained by load test. It should be noted that even with the use of

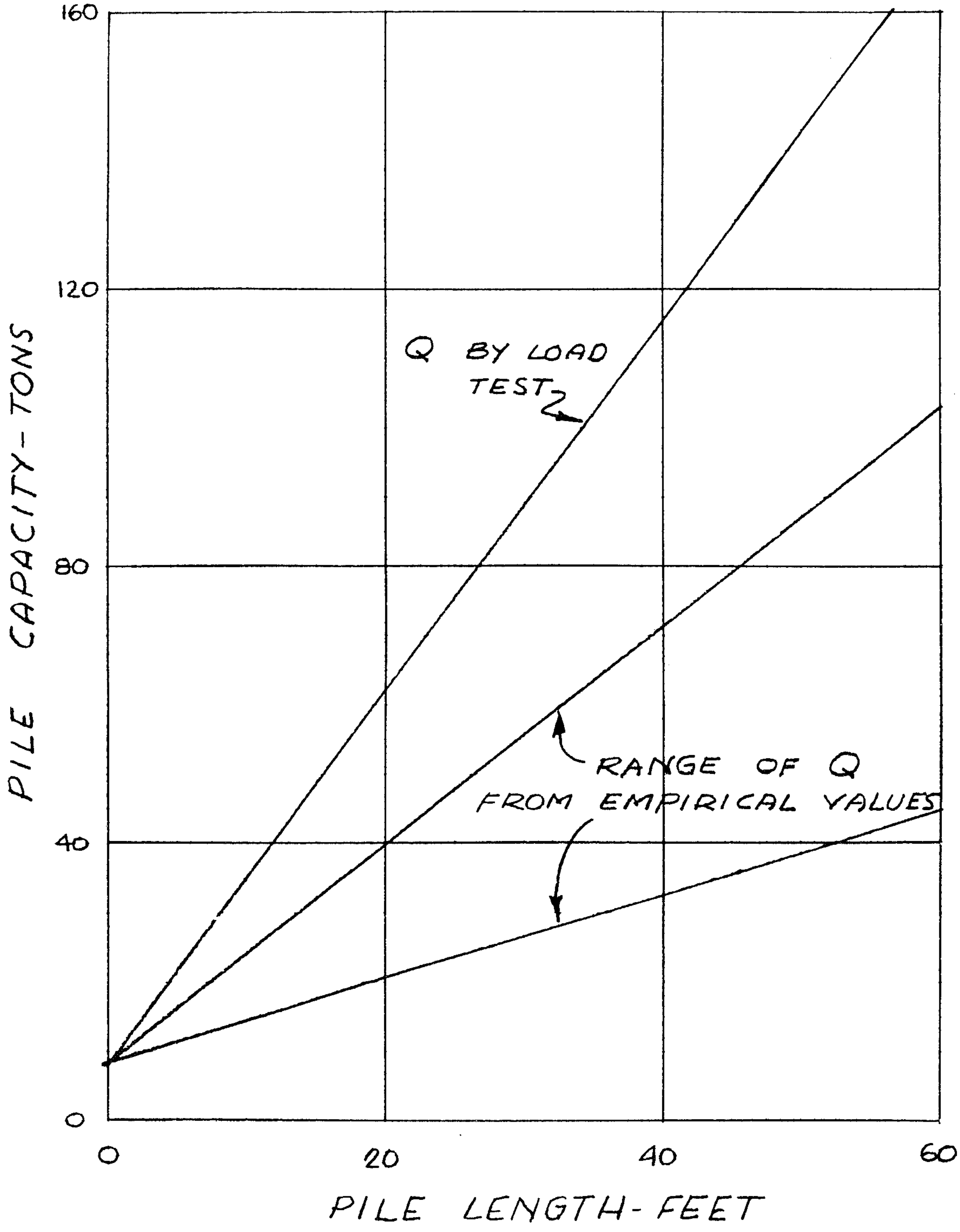


FIGURE 9-1

COMPARISON OF ULTIMATE PILE CAPACITIES--LOAD TESTS VERSUS EMPIRICAL VALUES the highest value from the table, the design would be ultra-conservative. The following results are taken from Figure 9-1 for a 40-foot pile.

By load test

$$Q = 115 \text{ tons}$$

From empirical values

$f_s = 1000$ p.s.f. $\quad Q = 70$ tons

$f_s = 400$ p.s.f. $\quad Q = 33$ tons

Thus, by using the load test results, the foundation cost could be reduced approximately 40 per cent over using a f_s value of 1000 pounds per square foot and approximately 70 per cent over using a f_s value of 400 pounds per square foot. As pointed out previously, these empirical values are presented only as aid to guessing and no claim is made as to their accuracy; Dr. Terzaghi says in this regard, ". . . tables serve only as a guide for making preliminary estimates. Reliable information cannot be obtained without performing loading and pulling tests on full-sized piles in the field" (reference 13).

Estimating the pile capacity on the basis of laboratory test, using $f_s = c$, gives similar conservative results, as shown on Figure 9-2. For a 40-foot pile, a saving of approximately 50 per cent could be realized by utilizing the results of field tests in lieu of the

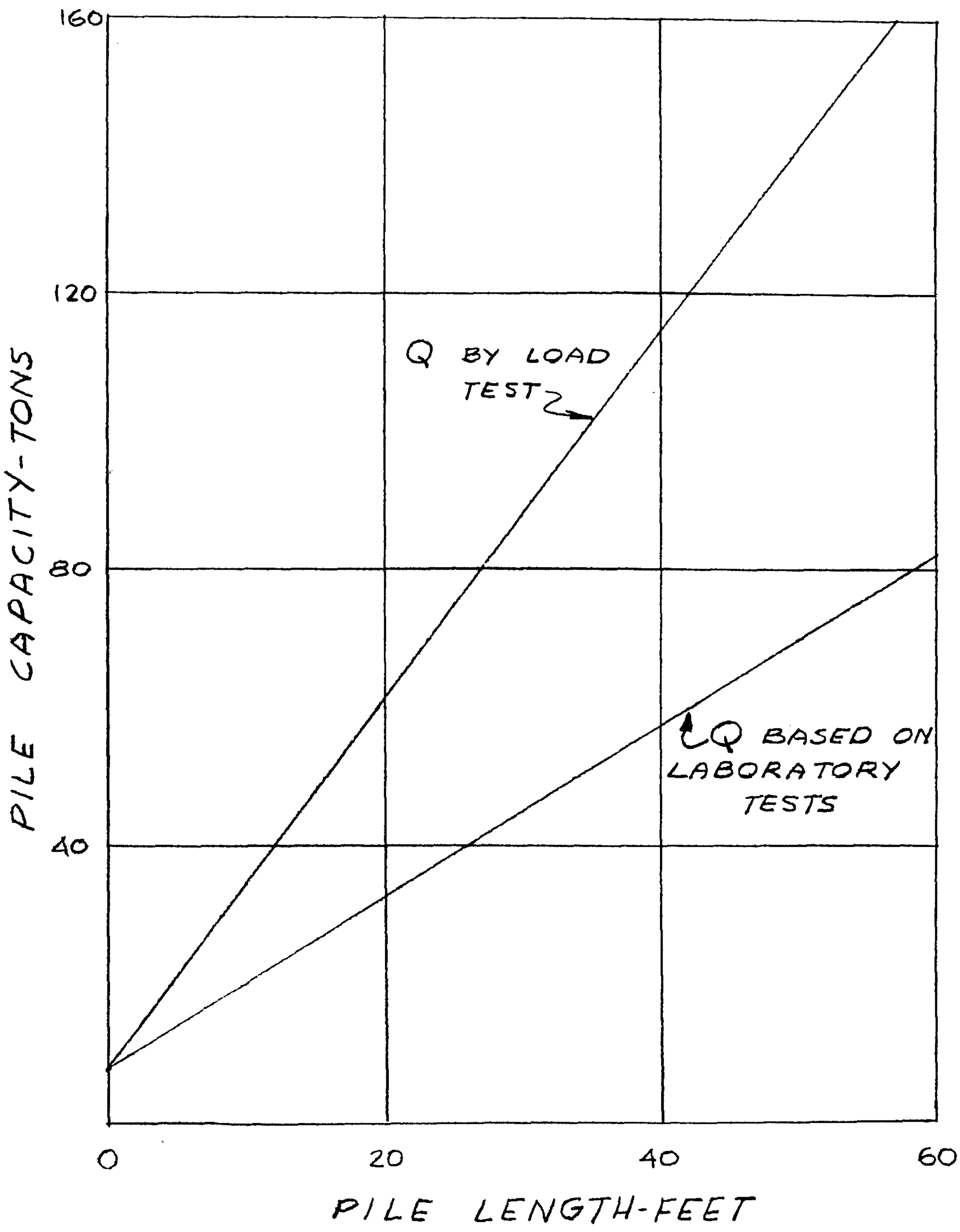


FIGURE 9-2

COMPARISON OF ULTIMATE PILE CAPACITIES--LOAD TESTS VERSUS LABORATORY TESTS constants determined by laboratory test. The estimated capacity was known to be conservative since bearing from cohesion was considered solely. Its difference in bearing capacities is easily explained by the soil's granular characteristics; that is, increased load is necessary to overcome friction between irregular grains to enable them to roll over one another.

It has thus been established that laboratory tests and theory are a great aid in evaluating the bearing capacity of a spread footing but practically worthless in estimating a pile's capacity.

CHAPTER X

INVESTIGATION OF ACCURACY OBTAINED BY EMPIRICAL PILE FORMULAS

This chapter should be unnecessary since soils theory clearly states that empirical pile formulas are not applicable to friction type piles. However, the author has noted that empirical pile formulas are in wide use in this general area; in fact, many pile concerns place great reliability on these formulas and are certain that NACA has erred somewhere by specifying that piles be driven to a certain depth rather than to a certain number of blows per foot.

The Committee on the Bearing Value of Pile Foundations, American Society of Civil Engineers, made an extensive study of the reliability of dynamic pile formulas. Work by Mr. Cummings revealed that all of the common formulas are illogical since they are based on an entirely unknown, and perhaps nonexistent, relation between dynamic and static pile resistance. The Committee generally concurred in the conclusion that no dynamic formula checked sufficiently against actual tests and experience to deserve the official approval of the Committee.

The Committee also concluded that no one formula gives consistently more accurate results than another. The

Engineering News Record Formula was deemed superior to the others on the grounds that was the simplest to use.

The Engineering News Record Formula for steam hammers is as follows:

$$Q_a = \frac{2WH}{S + 0.1}$$

where

Q_a (lb) = allowable bearing capacity of pile

W (lb) = weight of hammer

H (ft) = fall of hammer

S (in.) = penetration under the last blow

A comparison of the load test results in chapter VII with the noted capacities calculated by the empirical Engineering News Record Formula clearly shows the irrational behavior of such formulas. It has also been noted that capacity of pile as calculated by the ENRF is to a large extent dependent upon the size of the hammer used. On one large project on the base, the Gas Dynamics Laboratory, two different hammers were used. A pile driven by a Vulcan No. 2 consistently reached an ENRF indicated capacity of 20 tons at a depth of 40 feet while a pile driven by a Vulcan No. 1 just as consistently could only reach an ENRF capacity of 10 tons. This discrepancy is probably due to approximately the same energy being needed in both cases to overcome friction and losses which results in a greater net percentage left to do work in the case of the heavier hammer. There is no term in the ENRF, however, to reflect this item; thus, by judicious choice of a hammer, a contractor could seemingly obtain the required capacity with a shorter pile than would normally be required.

The ENRF has a theoretical safety factor of six but tests in the field have shown that the safety factor varies from less than 1 to more than 20 based on soil conditions and driving equipment.

At Langley Field, the safety factor, which would have been obtained by ENRF, has been observed to vary between the following limits:

Lower limit: Test Pile No. 2 - Aircraft Loads Building (Fig. 7-2)

$$\text{Safety factor} = \frac{\text{ultimate load by field test}}{\text{allowable load by ENRF}}$$

$$= \frac{60}{25} = 2.4$$

Higher limit: Test Pile - Water Tower Foundation (Fig. 7-16)

$$\text{Safety factor} = \frac{107}{8} = 13.4$$

Assuming that an average safety factor of about 8 is obtained for Langley soil by ENRF, where only a safety factor of 2.5 is adequate, approximately three times the number of required piles would be driven if the Engineering News Record Formula were used as a basis of pile capacity; thus, the foundation cost would needlessly be three times as great.

Empirical formulas are incompatible with modern soils theory in yet another way. These formulas, by specifying the number of blows per foot as a standard of when to cease driving, often cause a great variance in pile length throughout a project. It has been established that settlement is linearly inversely proportional to pile length for a given load. Thus, it naturally follows that the way to avoid damageable differential settlement is to drive all piles to the same depth providing, of course, that borings have revealed similar soil conditions.

Dr. Terzaghi sums up the case against pile formulas as follows: ". . . the use of the formula on the design of pile foundations is unsound on both economical and technical grounds. An exception to this statement can be made only if the cost of the load tests is greater than about one-half of the cost of the pile foundation."

CHAPTER XI

CORRELATION OF THEORY AND TEST RESULTS--CONSOLIDATION

It is recalled that theory predicted settlements of fairly large magnitudes due to primary consolidation which were not borne out by field measurements. As stated previously, no theoretical analysis is available which would enable the soils engineer to estimate secondary consolidation; it has to be observed in the field and rate of settlement extrapolated for estimating future settlements. Field readings to date indicate that settlement due to secondary consolidation, although negligible insofar as ordinary building structures are concerned, should be considered in the design of wind tunnels and similar research facilities where differential settlement of approximately 1/2 inch could be harmful. Field readings also indicate that great care should be exercised in the design for any foundation which is going to be subject to excessive vibrations.

The computation of primary consolidation is based on laboratory tests in which the attempt is made to duplicate actual field boundary conditions. It should be emphasized that unless the assumed hydraulic boundary conditions are in accordance with actual drainage conditions, the results of a settlement computation are not even approximately correct.

Such field conditions as a shallow continuous sand stratum, which could easily be missed by the borings, are sufficient to cause accelerated drainage and make the computations incorrect. Thus, only in a very few actual cases, is it possible to make an accurate settlement estimate; the usual practice is to make conservative assumptions such that an upper limit for settlement is obtained and provided for in the design.

It is probable that the soils' engineers at both the Haller Testing Laboratories and the Waterways Experiment Station made theoretical settlement computations similar to computations made in chapter VI because, in both cases, settlements of approximately the same magnitude were estimated; field readings on these projects also failed to record any appreciable primary consolidation.

Various soil laboratories have found it extremely difficult to classify the characteristics of the grey silty sand as either "granular" or "cohesive." By grain size, the soil is definitely "granular"; however, tests reveal that the grains are separated by a thin film of clay and silt so that, with no normal load acting, the soil exhibits "cohesive" properties only. It is interesting to note that the soil study conducted by the Haller Laboratories considered the soil "granular" while those conducted by the University of Michigan considered the soil "cohesive."

A probable explanation of the discrepancy between fact and theory, regarding primary consolidation, follows from the above. It is that, in some manner, this film between grains is broken causing the sand grains to come into contact and, thus, to resist consolidative settlement by friction. This film could be broken in any manner which would tend to compact the soil, that is, placement of an external load or displacing soil by pile driving.

Some substantiating evidence is available to strengthen this theory. On the Landing Loads track, close measurements of heave due to pile driving were recorded; although heave of a small magnitude was noted, it was obvious that most of the soil was displaced laterally reducing the voids in the surrounding soil. Thus, it would seem to follow that high displacement pile would be most satisfactory for local soil conditions. An examination of the load tests in chapter VII reveals this to be a fact; it is seen that the high displacement Raymond taper pile tested superior to the cylindrical type. The author feels that the high capacities obtained on piles from load tests are sufficient proof that the film is broken and friction between grains becomes paramount, that is, that the soil becomes primarily granular in action. Further evidence that soil acts as a sand is available from the time-settlement readings of the Freon compressor foundation (Fig. 8-3). Increased settlement was noted during operation of the compressor; similar vibratory methods are often used to compact sand while having negligible effect on cohesive soils.

It is regretted that, of necessity, the soil samples tested in the laboratory are taken prior to pile driving disturbance and, therefore, do not represent actual field conditions. However, it is believed that the discrepancy between the theory predicted settlement and field readings can best be explained by the film between grains being broken rather than failure to duplicate field boundary conditions in the laboratory.

CHAPTER XII

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The following results have been established in this thesis for the grey silty-sand stratum at Langley Field, Virginia.

(1) Various properties of the grey silty sand were determined by laboratory test. The most important and useful are the soil's cohesion value and its angle of internal friction. It was determined that their average values are:

$$c = 0.33 \text{ tons per square foot}$$
$$\phi = 24^\circ$$

(2) For the design of spread footings, theoretical analysis gives a sufficiently accurate estimate of a footing's ultimate bearing capacity to warrant reliability. Theoretical analysis estimates a plate's bearing capacity with no surcharge as 11 tons per square foot; in a test, the soil actually failed at 8 tons per square foot.

(3) Neither theoretical analyses nor dynamic pile formulas gives a sufficiently accurate estimate of pile's load-carrying capacity to warrant their use. Both methods consistently underestimate a pile's capacity by at least 100 per cent.

(4) Field records on several projects reveal that primary consolidation is negligible despite the opposing predictions obtained by usual soil's theory. Settlement readings have failed to indicate any settlement due to primary consolidation even where theory estimates settlement of approximately 4 inches.

(5) Two warning signals are noted from the field settlement readings; these are (1) secondary consolidation is not negligible when 1-inch settlements are critical and (2) foundations may settle seriously if subjected to continual vibration.

From these results, it is concluded that neither theory nor test results are sufficient alone but must be used integrally by an engineer experienced in the assumptions involved. The logical conclusion reached is to use the theoretical analysis for spread footing design, load test results for pile design, and some combination of theory and field results for settlement analysis. However, in each case, a knowledge of soils is necessary to determine or estimate the soil constants, to interpret test results correctly, and to choose an applicable safety factor. The items influencing the soil engineer's decisions will be discussed more thoroughly in the following paragraphs pertaining to the author's recommendations for future foundation design on Langley Field, Virginia.

On any future projects of a large magnitude in which the utilization of spread footings is planned, a small soils investigation should suffice. This soils investigation should contain the following items:

(1) Approximately five borings, 50 feet deep

(2) Sieve analysis of samples at 10-foot intervals

(3) Tri-axial shear tests, two per hole

The borings are necessary to ascertain the continuity of the soil stratum. The gradations will serve as checks for the comparative data of gradations contained in chapter IV. From the tri-axial shear tests, the engineer can obtain design values of cohesion (c) and the angle of internal friction (ϕ). It is then recommended that the designer use the theoretical analyses in chapter V as design procedure. It is urged that the minimum safety factor of 2.5 be applied to obtain allowable bearing capacities.

For spread footings on small projects where the cost of even the above outlined soils investigation would be unwarranted, it is recommended that the average values of c and ϕ contained in the summary be used in conjunction with the theory in chapter V for design. However, due to the lack of a substantiating soils investigation, a safety factor of 3 should be applied.

Since bearing capacity and the magnitude of settlement both vary with respect to pile length, it is necessary in specifying a pile length to ascertain its adequateness on both counts. Another factor influencing the allowable bearing capacity of a pile is the material of the pile itself. The following table (reference 13) recommends the following allowable loads for various type piles.

Type of pile	Allowable load (tons)
Wood	15-25
Composite	20-30
Cast-in-place concrete	30-40
Precast reinforced concrete	30-45
Steel H-section	30-45

General procedure to date at NACA when a pile foundation is required is to choose on the basis of economy between a 40-foot, 15-ton design capacity, timber pile and a 30-foot, 30-ton design capacity, large displacement tapered concrete pile. These two types and lengths of piles have proved satisfactory in that settlement has been negligible. Although the safety factor against failure in both cases is ultra-conservative, care must be exercised in revising the pile lengths and/or allowable capacity to guard against excessive settlement.

The author recommends, however, that a continual attempt be made to lower the safety factor by increasing the allowable bearing capacity and/or decreasing the pile length. The revisions should be tested first on buildings of ordinary construction where a small additional settlement would be harmless. If settlement does not become excessive, the revised allowable loads and/or lengths could then be applied to the wind-tunnel foundations. It is recommended that the following steps be followed:

For Concrete Tapered Piles

At present	- 30 feet	30 tons capacity
Step 1	- 30 feet	40 tons capacity
Step 2	- 25 feet	40 tons capacity

For Wood Piles

At present	- 40 feet	15 tons capacity
Step 1	- 40 feet	25 tons capacity
Step 2	- 30 feet	25 tons capacity

Based on past tests, it is believed that the capacity and length stated in step 2 of each type pile will satisfy the safety factor requirements; needless to say, this should be checked by load tests on each project. Accurate level readings are necessary to determine when and if it is feasible to progress from present status to step 1 and from step 1 to step 2. To the author, it appears quite likely that step 2, in each case, will prove satisfactory.

Since it seems logical to assume that the same factors which influence primary consolidation will influence secondary consolidation, the following procedure is recommended for settlement analyses. First, make a complete settlement analysis based on theory contained in chapter VI; in this manner, the theoretical settlement of each footing will be calculated and tabulated. To bring these theoretical settlements in line with actual field results with a safety factor of two, the second step is to arbitrarily revise the maximum theoretical settlement to 2 inches and to prorate all others accordingly; these settlements should be termed "preliminary design settlements."

These preliminary design settlements should be referred to the structural engineer involved for his study. If the structural engineer's study reveals that these settlements will not place undue stress on the structure, these settlements could be designated "final design settlements." If the structural engineer finds that the differential settlement is excessive, the footings should be redesigned to reduce the differential settlement, the settlement recalculated and prorated, and a new set of "preliminary design settlements" forwarded to the structural engineer.

The above recommendations are based partly on theory, partly on test results, and partly upon experience and common sense. It is believed that the incorporation of these recommendations in future design will affect a saving to the Government as well as resulting in sound technical design.

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